JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 7. 1940–41.

JUNE 1941.

ROAD ENGINEERING SECTION MEETING.

22 April, 1941.

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Chairman of the Section, in the Chair.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of the Section were accorded to the Author.

Road Paper No. 3.

"Road Traffic Calculations." †

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TABLE OF CONTENTS.

	PAGE
Introduction	247
Maximum density	249
Effect of speed upon traffic capacity	250
Cycle-time and volume of traffic at controlled intersections	253
Capacity of roundabouts and one-way streets	257
Delay at intersections	259
Random series	261
Standing vehicles	261
Conclusions	262
Acknowledgement	262

INTRODUCTION.

WHEN road traffic was light and the methods of dealing with it were simple, there was no need to make calculations; but now that it is necessary to

† Correspondence on this Paper can be accepted until the 15th August 1941, and will be published in the Institution Journal for October 1941.—SEC. INST. C.E.

regulate dense volumes of traffic, and various alternative methods of control are available, calculations can be a great aid to judgment. As in other branches of engineering practice, the blind use of formulas and Tables cannot be substituted for sound judgment, but a decision formed as the result of calculations based upon the best available data is more likely to be correct than is one based upon general impressions. The methods in use have been developed to solve problems as they have arisen in practice, principally by the engineers of the Ministry of Transport and the traffic-signal makers in Great Britain, and by corresponding engineers in the United States of America.

Statistics.

The principal data upon which traffic calculations are based are traffic statistics, just as, for instance, calculations for drainage and water-supply are based upon statistics of rainfall and population: but statistics are useless until interpreted properly.

Traffic statistics are best taken at road intersections, and it is more important to separate the traffic into its separate directions than to have a very detailed classification into types. Division into—

(1) omnibuses;
(2) heavy motor-vehicles;
(3) light motor-vehicles;
(4) slow vehicles;
(5) bicycles;

is sufficient. A short count taken on this basis with careful choice of period can be of more use than a census extending over a long period which gives only the total volume of traffic; for example, the investigations for the successful Trafalgar Square one-way traffic scheme were based upon traffic-counts taken single-handed in half a day.

In some instances observations of standing vehicles are also required, and these are best recorded as the mean of a number of observations of the vehicles standing in the street, taken at intervals during the period under consideration.

Nature of Traffic Flow.

Anything of the nature of a hydraulic analogy is misleading in dealing with traffic-flow, since traffic consists of a number of individual units, the movement of each of which is limited by certain arbitrary rules and by dynamic limitations, but otherwise is governed by the drivers' will. It has been shown that the passing of vehicles in a freely-flowing traffic stream may be taken as a random series [1]. Vehicles moving in a stream as close

[1] W. F. Adams, "Road Traffic Considered as a Random Series", Journal Inst. C.E., vol. 4 (1936–37), p. 121 (November 1936).

as their drivers consider safe may be regarded as moving at uniform intervals in each lane, although the actual spacing is a chance one within narrow limits.

Units and Definitions.

The units generally used are: (1) vehicles per hour, for traffic volume, and (2) vehicles per hour per lane, for density, as these furnish numbers of convenient magnitude; they might conveniently be expressed by the abbreviations v.p.h. and v.p.l. Speeds are usually expressed in miles per hour, but vehicles per second and feet per second are sometimes more convenient for formulas. A "stream" of traffic is all the traffic on a definite route, for example, that turning left at an intersection; "opposite" or "complementary" streams are those on the same route, but in opposite directions. The number of "lanes" of traffic a road can carry is the number of vehicles which can move abreast or pass upon it; in a two-way road this may be an uneven number if there are no central obstructions. The actual width of a lane will vary with the speed and the types of vehicles, and may be less in existing roads than would be considered desirable in design of new work.

The present Paper is limited to the consideration of the flow of traffic and of delays therein, and does not deal with other calculations necessary in connexion with design of roads for modern traffic, as it is intended solely to give a summary of the basis of traffic calculations.

Maximum Density.

Counts taken at places where the traffic volume is clearly at or very near the maximum which the road can carry at that point, exhibit considerable variation, even when allowance is made for width. A peak amounting to 989 vehicles per hour per traffic-lane has been recorded on the Victoria embankment at Northumberland Avenue, whilst at times more than 1,000 vehicles per hour leave the narrow gateway at Hyde Park Corner. On the other hand, the peak figure for Whitehall at Trafalgar Square was 550 vehicles per hour per traffic-lane, although this road was clearly working to the limit of available capacity. The traffic in Broad Sanctuary at Parliament Square had practically the same density at the peak, namely 540 vehicles per hour per lane, but the road presented every appearance of having a wide margin of capacity. Presumably these differences are due to (1) variations in the speed and composition of traffic; (2) the hold-up at the intersections.

The traffic is passing through a controlled intersection for only part of the time; when vehicles are running as close together as possible the traffic is flowing at a rate called the "saturation density" (S).

The effect of the stoppage may be expressed as a factor usually designated the "delay factor", which is the sum of the effective running periods in

any interval divided by the total time; it would perhaps be better to call this factor the "intersection factor." The effective running time is the "go" period, less an allowance for the delay in starting up, which experience shows can be taken at, say, 6 seconds; for example, in a road given 20 seconds green signal out of a cycle of 50 seconds, the factor—

$$= \frac{20 - 6}{50} = 0{\cdot}28.$$

In the case of Whitehall, the intersection factor was observed to be 0·366. The density of traffic divided by this factor—

$$= \frac{550}{0{\cdot}366} = 1{,}500.$$

It is possible to measure the "saturation density" by counting the traffic for the short periods during which it is running at its densest.

Observation of the London-bound traffic on the Portsmouth road just clear of the controlled intersection in Esher on the evening of August Bank Holiday, 1939, revealed variations in the density during the different "bursts", but a general tendency for the number of vehicles to be proportional to the time. There was room for one lane of traffic in each direction, and the average density during the actual "bursts" of traffic was equal to 2,000 per hour, excluding bicycles. The traffic was composed of 85 per cent. light cars, 3 per cent. omnibuses, and 12 per cent. bicycles. The speed was about 20 miles per hour at the point of observation. This density is higher than is usually found, even with light traffic. One vehicle per second (3,600 vehicles per hour) in a single lane was observed in bursts of about 10 seconds' duration on the Croydon by-pass; but this was at a temporary bottle-neck, causing two lanes of traffic suddenly to converge into one. The conditions appeared to be very unsafe.

Numbers of other observations have been made by both methods, giving results generally ranging from 1,200 to 1,800 vehicles per hour for town conditions. For practical purposes the best way to obtain a working value is by observations of the delay factor and density of an actual intersection where conditions are similar.

Effect of Speed on Traffic Capacity.

Whilst it is possible to observe the actual density of dense traffic-streams as described above, it is difficult to make observations of the relation between speed and the maximum safe density. Clearly the maximum density is obtained when there is no overtaking. If all of the vehicles are travelling at the same speed (v feet per second) the maximum number of vehicles which can pass in a given time with safety must depend upon the minimum safe headway, in seconds, between one vehicle and the next. This, in turn, also depends upon the speed and upon the vehicle and driver.

Attempts to measure the distances between moving cars at various speeds have not yielded consistent results.

If it is considered sufficient for a vehicle to be able to stop without colliding with the vehicle in front when that stops, the headway will be the time taken to apply the brakes on one car after those of the preceding car have been applied, plus the time required to travel the length of the car (L). Experiments have shown that when "stop" lights are fitted to cars a mean of 1 second is reasonable for the time taken to apply the brakes after the "stop" light has appeared. When no "stop" light is used more time is necessary. The headway required is therefore—

$$1 + \frac{L}{v} \text{ seconds.}$$

The number of vehicles per second

$$= \frac{1}{1 + \frac{L}{v}},$$

and the saturation density,

$$S = \frac{3{,}600}{1 + \frac{L}{1{\cdot}47V}} \text{ vehicles per hour,}$$

where V denotes the speed, in miles per hour.

Multiplying top and bottom by $1{\cdot}47V$,

$$S = \frac{5{,}280V}{1{\cdot}47V + L} \quad \ldots \ldots \quad (1)$$

Line A on *Fig. 1* shows this plotted for light cars ($L = 15$ feet). Densities conforming to higher speeds on this curve have been observed in certain cases, but they appear to be dangerous, and it is usually considered that the braking capacity of the vehicle is involved.

The distance required to pull up after brakes have been applied varies as the square of the speed; the following expression makes allowance for this:—

$$S = \frac{5{,}280V}{L + AV + \frac{V^2}{B}} \quad \ldots \ldots \quad (2)$$

Results in accord with practical experience are obtained when A = 1·47, and B = 30 for light vehicles and 20 for heavy vehicles.

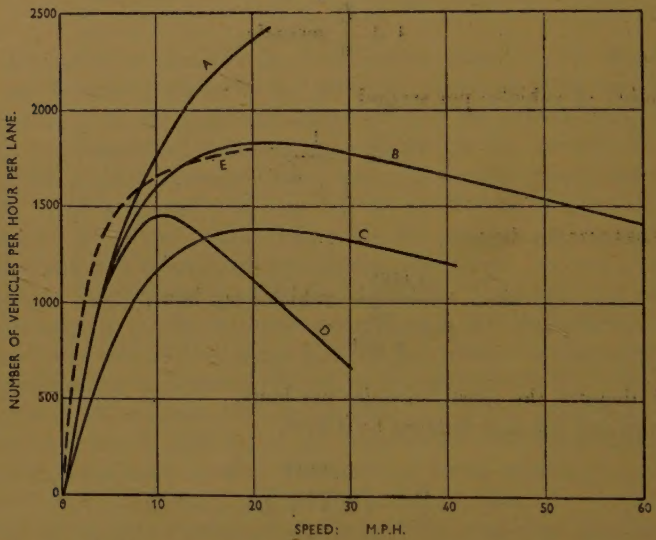
The denominator of the right-hand side of equation (2) represents the headway, in feet; this is not the full pulling-up distance, as consideration will show to be reasonable, since any cause for stopping will be visible in

front of the forward car. Equation (2) is plotted on *Fig. 1* as curve B for light vehicles and curve C for heavy vehicles.

It will be seen that this basis gives the maximum capacity at a speed of between 20 and 25 miles per hour, which is usually known as the economic speed—economic, that is, from the point of view of the highway authority, and not that of the motorist.

Formulas of this type are generally accepted, although the constants vary; some authorities, however, adopt a value of 2·2 for the power of V, instead of 2, whilst others omit the factor involving the first power of V.

Fig. 1.



The type of vehicle has a relatively small effect, provided it can maintain the same speed as the rest of the traffic. Slow vehicles which have to be overtaken by others present a different and more complicated problem, but at worst they can only reduce the capacity to that indicated by their speed. A bad condition of the road-surface can quite seriously affect its safe capacity, as is shown by curve D, which represents very bad conditions, with a braking coefficient ranging from 0·4 at 0 miles per hour to 0·09 at 30 miles per hour, as suggested by Messrs. R. G. C. Batson, G. Bird, and R. E. Stradling [1].

The capacity of cycle-tracks may be dealt with on the basis of equation (1), as a cyclist does not obstruct the view of a following cyclist; but, as no stop light is fitted, a higher value should be allowed for the time required to apply brakes.

[1] "Road Engineering Problems: Judging the Slippery Road", Journal Inst. C.E., vol. 2 (1935–36), p. 443 (April 1936).

Number of cyclists per lane per hour

$$= \frac{3{,}600}{1{\cdot}8 + \dfrac{6}{1{\cdot}47 V_c}} \simeq \frac{3{,}600}{1{\cdot}8 + \dfrac{4}{V_c}} \quad . \; . \; . \; . \; . \; . \quad (3)$$

This is illustrated by curve E on *Fig. 1*.

Whilst the densities given above are in accordance with experience, and the average spacing between vehicles is theoretically safe, the actual spacing between vehicles, even when running at maximum density, varies considerably, and many drivers travel closer to the car in front than is actually safe, as is shown by the high proportion of "head-to-tail" collisions. Observations made on the Portsmouth road in connexion with those referred to above, showed variations ranging from $\frac{4}{5}$ second upwards, with a mean of 1·28 second (if the values in excess of 3 seconds were ignored); the variation about the mean appears to be a chance one.

Probably the reduction in speed experienced with dense traffic in towns is due not to simple factors such as those mentioned above, but to frequent intersections, standing vehicles, omnibus-stops, pedestrians, etc.

The capacity of a two-way road wide enough for an odd number of traffic-lanes should be taken as that of the next lower even number of lanes, because the odd lane is usable only for overtaking.

Cycle-Time and Volume of Traffic at Controlled Intersections.

It is desirable to determine how the most suitable cycle-time varies with traffic-volume. The most efficient cycle is the shortest which will handle the traffic; although a longer cycle would enable a number of vehicles to pass which would otherwise be stopped, it would stop a similar number of vehicles which would otherwise pass, and the delay would be increased.

Unless the number of vehicles is small, they will eventually be entering the junction at a uniform rate per traffic lane. This rate is the saturation-density, and the phase should end as soon as the density falls off to any extent. For each road the average number of vehicles per lane in each cycle $= \dfrac{cD}{3{,}600}$, where D denotes the density of traffic entering by that road, and c denotes the cycle-time, in seconds.

The time required to pass this traffic $= a + \left(\dfrac{cD}{3{,}600} \div \dfrac{S}{3{,}600}\right)$ where a is a constant number of seconds to allow for the time required to reach saturation and to allow a margin for variation in the number of vehicles arriving per cycle.

Similarly, for each phase, the length of the phase is governed by the

road running during that phase which has the greater density; thence for a two-phase cycle :—

$$c = a + \frac{cD_1}{S} + a + \frac{cD_2}{S}$$

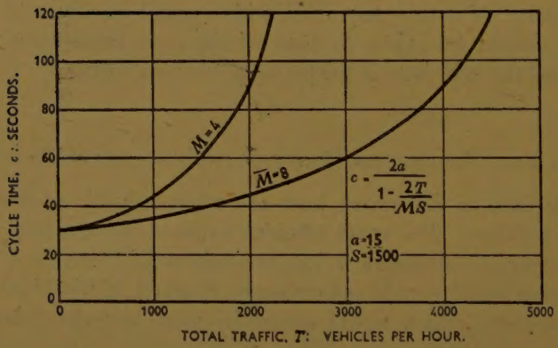
$$\therefore c - \frac{c}{S}(D_1 + D_2) = 2a$$

$$\therefore c = \frac{2a}{1 - \dfrac{(D_1 + D_2)}{S}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Similarly, for the general case of a multi-part cycle of n phases with a dead period (such as an "all-red" period) of r seconds :—

$$c = \frac{na + r}{1 - \dfrac{D_1 + D_2 + \ldots + D_n}{S}} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Fig. 2.



If (i) the densities of traffic on all the streets are nearly equal, or (ii) the densities on opposite roads are nearly equal, and the roads have equal numbers of traffic-lanes, equation (4) may be simplified to :—

$$c = \frac{2a}{1 - \dfrac{2T}{MS}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

where T denotes the total traffic entering the intersection and M denotes the number of lanes of traffic passing through. Curves showing the variation of cycle time with total traffic, for two-lane and four-lane roads, are reproduced in *Fig 2.*

The values of a and S are determined empirically; in fact, equation (4a) was determined empirically before equations (4) and (5) were evolved.

The value of a may be said to be equal to the minimum "go" period plus the amber period, usually about 14 seconds, whilst the value of S will vary with the traffic conditions and the composition of the traffic, and will range from 1,200 to 1,800 vehicles per hour per lane. The traffic census at the peak hour should be used if available; otherwise a lower value should be adopted for S, in order to allow for variation between peak and average.

A value of S obtained as on p. 251, *ante*, may be adopted, but equation (4) may be transposed as follows to calculate the value at an actual intersection where the density of traffic and the cycle are known:—

$$c = \frac{2a}{1 - \frac{(D_1 + D_2)}{S}}$$

$$\therefore \frac{D_1 + D_2}{S} = 1 - \frac{2a}{c}$$

$$\therefore S = \frac{D_1 + D_2}{1 - \frac{2a}{c}} \quad \ldots\ldots\ldots\ldots \quad (6)$$

Similarly, for the general case,

$$S = \frac{D_1 + D_2 + \ldots + D_n}{1 - \frac{na + r}{c}} \quad \ldots\ldots \quad (6a)$$

These expressions, in addition to enabling a suitable cycle for fixed-cycle signals to be determined, may be used to estimate the average cycle for vehicle-operated signals and for police control when the traffic is heavy. In the former case a should be made equal to the minimum "go" period plus amber plus 2 seconds. Police tend to use longer cycles than are desirable in the case of signals. The formulas may also be used to fix a suitable value of the "maximum" period for vehicle-actuated signals.

Capacity of Controlled Intersections.

An approximate formula for the total capacity of an intersection may be obtained as follows:—

$$c = \frac{2a}{1 - \frac{2T}{MS}} \quad \ldots\ldots\ldots\ldots \quad (4a)$$

$$\therefore \frac{2T}{MS} = 1 - \frac{2a}{c}$$

$$\therefore T = \tfrac{1}{2}\left(1 - \frac{2a}{c}\right)MS \quad \ldots\ldots\ldots \quad (7)$$

This represents the maximum capacity of a simple intersection under ideal conditions.

The capacity is usually reduced in practice by inequalities in the distribution of turning traffic, the tidal effect and the widths not being proportional to the traffic.

The factor $\frac{1}{2}\left(1 - \frac{2a}{c}\right)$ is a measure of the efficiency of the intersection in comparison with an ideal (flyover) intersection. A suitable maximum for c is 90 seconds, when the efficiency equals $\frac{1}{2}\left(1 - \frac{28}{90}\right) = 0{\cdot}345$.

To calculate the capacity of a more complicated intersection, a suitable total cycle (say 30 seconds per running phase, to give short delays) should be assumed, whence

$$\left(1 - \frac{na + r}{c}\right) \text{ may be calculated.}$$

Transposing (6) and (6a),

$$D_1 + D_2 = S\left(1 - \frac{2a}{c}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$D_1 + D_2 + \;.\;.\;.\; + D_n = S\left(1 - \frac{na + r}{c}\right) \quad . \quad . \quad . \quad (8a)$$

If the density of traffic on all roads but one is fixed, the density, and thence the capacity of the remaining road may be found. Alternatively, if the proportions of traffic be known, the total capacity may be calculated.

Bottlenecks.

A controlled "bottleneck" may be treated in a similar manner. The necessary clearing period (r) will be the time required to cover the total length of the one-lane section at normal speed. This will include one red/amber period, so that the value of a may be smaller than in the case of an intersection. Then

$$c = \frac{2a + 2r}{1 - \dfrac{D_1 + D_2}{S}}$$

If the traffic in the opposite directions is equal,

$$D_1 = D_2 = D.$$

A reasonable maximum cycle may be taken as—

$$2(b + r)$$

where b denotes a reasonable maximum "go" period: in that case

$$c = 2b + 2r = \frac{2a + 2r}{1 - \frac{2D}{S}}$$

The total capacity is

$$2D = \left(1 - \frac{2a + 2r}{2b + 2r}\right)S$$

$$\therefore\ 2D = \left(1 - \frac{a + r}{b + r}\right)S \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Capacity of Roundabouts and One-Way Streets.

Considering first the small roundabout or safety-island lay-out where the traffic-streams cross almost at right angles, and adopting a basis similar to that used in equation (2), the headway, in seconds, before a vehicle Y arrives at the path of a crossing vehicle X, is equal to $1 + \frac{v_y}{B_1}$ seconds, where B_1 is a constant.

The time required to clear the width of the path of Y is $\frac{L + W}{v_y}$ seconds, where W denotes the width of the vehicle.

The headway required by vehicle X is $1 + \frac{v_x}{B_1}$ seconds, and the time required by X to clear the path of Y is $\frac{L + W}{v_x}$ seconds.

The total headway in the stream in which Y is travelling is

$$1 + \frac{v_y}{B_1} + \frac{L + W}{v_y} + 1 + \frac{v_x}{B_1} + \frac{L + W}{v_x} \text{ seconds.}$$

If $v_y = v_x = v$, as is normally the case, the headway is

$$2\left(1 + \frac{v}{B_1} + \frac{L + W}{v}\right) \text{ seconds.}$$

It will be seen that this is twice the headway in a normal stream, but with $(L + W)$ instead of L. The capacity of the intersection of the two streams may be taken as

$$T_R = \frac{5{,}280V}{L + W + 1{\cdot}47V + \frac{V^2}{B}} \quad . \quad . \quad . \quad . \quad (10)$$

where V denotes speed, in miles per hour.

The turning traffic which does not cross other traffic at that point is carried in addition. V is about 15 miles per hour.

For example, comparing the capacity of a junction of two roads, each having a 20-foot carriageway with symmetrical traffic, one-third of which turns left or right, (1) signal-controlled; (2) controlled by a small roundabout:—

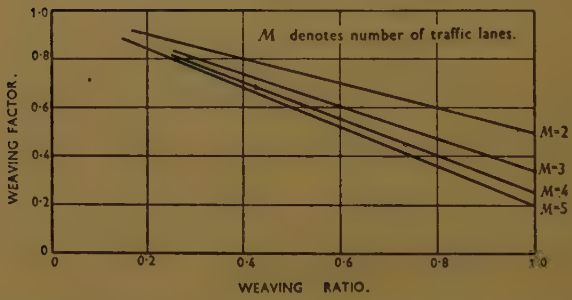
(1) The capacity of the signal-controlled intersection will be given by

$$T_S = \tfrac{1}{2}\left(1 - \frac{2a}{c}\right)MS \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

$$= \tfrac{1}{2}\left(1 - \frac{28}{60}\right)4 \times 1{,}800$$

$$= 1{,}930 \text{ vehicles per hour.}$$

Fig. 3.



(2) The capacity of the intersection of the streams at one corner of the roundabout is

$$T_R = \frac{5{,}280 \times 15}{15 + 5 + 1{\cdot}47 \times 15 + \dfrac{15^2}{30}},$$

$$= 1{,}600 \text{ vehicles per hour.}$$

Half of this number, plus the left-turning traffic, represents the traffic entering from each road; thus the total capacity is

$$4\left(\frac{1{,}600}{2}\right) \times \frac{6}{5} = 3{,}850 \text{ vehicles per hour.}$$

The small roundabout has not always twice the capacity of a signal-controlled intersection. For example, in this case, if the roads were double the width, the signals would have double the capacity, but the capacity of the roundabout would remain unaltered.

In the case of one-way streets or roundabouts with adequate weaving-length the intersecting streams are moving more nearly in the same direction, and as the weaving-length increases the capacity approaches that of a free-running stream. *Fig. 3* shows suggested empirical values of a "weaving factor" which, it is suggested, might be used in the same

way as the intersection-factor. This is given in terms of the "weaving ratio", that is, the minimum width of the road divided by the effective weaving-length and the number of lanes M. The difficulty of checking any such factor with actual densities lies in the fact that few, if any, roundabouts are working to capacity, except where they are also affected by traffic cuts, as, for example, at Trafalgar Square.

If opposite streams of traffic are equal the traffic on any side of a roundabout is equal to half the total traffic entering; this is obvious in the case of a direct crossing without turning traffic, but may be shown to be of general application.

Delay at Intersections.

Whilst it is not possible to state what an individual driver will experience at an intersection, the effect upon the mass of traffic can be estimated by calculating the proportion which will be stopped and the average delay. Certain assumptions are necessary, and these may lead to some degree of inaccuracy; but if similar assumptions are made in different cases results of some comparative value may be obtained.

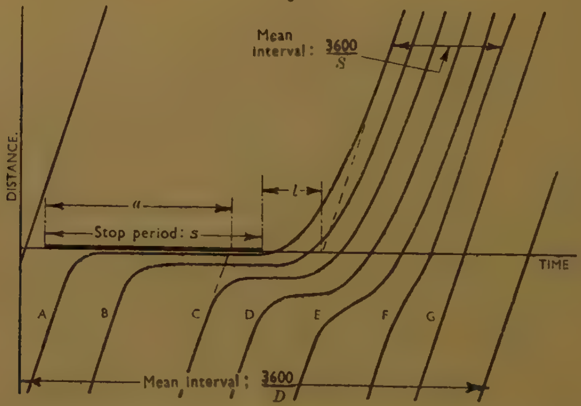
The fixed-time signal control presents the simplest case. If the red or red/amber signal is shown to a road for s seconds out of a total cycle c, a proportion $\frac{s}{c}$ of the vehicles on that road will, on the average, receive a "stop" signal. The fact that some will slow up on approaching, so as not to reach the intersection until the green signal is given does not alter the fact that they are delayed. Therefore if a sufficiently long period were taken into account, $\frac{s}{c}$ of the vehicles on that road would receive a "stop" signal. For each vehicle delayed there will be a stop of up to s seconds, but as it is a matter of chance at what point in the red period a vehicle arrives, the average delay-period will be $\frac{s}{2}$. On that basis the average delay to all vehicles is $\frac{s^2}{2c}$; this has usually been taken as a sufficient estimate of the delay, but it is on the low side.

A loss of time occurs in accelerating to normal speed; this will vary considerably, but a reasonable average value can be assumed. An average car will accelerate through the gears from standstill to 20 miles per hour in about 10 seconds. If it had not been stopped it would have taken only half that time to travel the same distance; therefore the loss of time $l = 5$ seconds. This is a sufficient allowance if only one vehicle is stopped per traffic-lane, but with more traffic, vehicles not directly affected by the "stop" signal may be delayed by those in front. This is illustrated in *Fig. 4.*

A method of allowing for the extra delay is given in the Appendix.

Vehicle-actuated signals or police control present a more complicated problem. For very light traffic on the road considered, the proportion stopped will be equal to the probability of there being a vehicle on the other road during a period (the " vehicle-interval " period) immediately preceding the arrival of a vehicle on the road considered. The average delay is the time of waiting for such a gap, plus the red/amber period and plus the time required to attain normal speed. Adopting the method devised by Mr. W. F. Adams, Assoc. M. Inst. C.E.[1] when considering the question of

Fig. 4.



pedestrians crossing the road, and assuming normal traffic on the other road, the proportion of vehicles delayed is equal to $(1 - e^{-Ni})$, and the mean delay to these vehicles is

$$1/Ne^{-Ni} - i/(1 - e^{-Ni}) + 3 + l,$$

where N denotes the number of vehicles per second on the other road, i denotes the vehicle-interval for that road, the amber period is 3 seconds, and l denotes the loss of time in accelerating.

For very heavy traffic the average delay is slightly less than with well-adjusted fixed-time signals ; but the methods suggested above may be used for most purposes of design.

For intersections where the " slow " or " halt " sign is used, methods similar to those suggested above for light traffic under signal control are suitable. It is then necessary to know the minimum gap required to cross in safety ; further investigation is required, but it is probably of the order of 5 seconds for the " slow " sign and 7 seconds for the " halt."

Before these methods can be accepted as reliable they must be checked against observations of delays at actual intersections. The most reliable

[1] *Loc. cit.*

method would appear to be to drive through the intersection several times, noting the number of seconds taken to pass between two points far enough away from the intersection not to be affected by the conditions there, but not so distant as to be affected by other circumstances. The shortest time can be taken as the normal. The Author's experience is that to observe the delay to vehicles from the ground is not practicable when the numbers are large.

Cost of Delays.—The total cost to the community of traffic delays may be expressed in vehicle-hours in a chosen period, such as a year. A monetary value may be ascribed to a vehicle-hour, based upon the cost of running the vehicle and the value of the time of drivers and passengers. Doubt is sometimes thrown upon the value of such estimates; but it should be appreciated that the saving of say, 1 vehicle-hour per hour means that one vehicle fewer is needed to carry the passengers or goods.

Random Series.

Mr. W. F. Adams [1] has demonstrated how the laws of random series may be applied to road traffic problems, whilst Dr. F. Garwood [2] has also recently developed a method of calculating the probability of vehicle-actuated signals running to the maximum.

Standing Vehicles.

The effect of standing vehicles upon moving traffic does not admit of simple analysis. Whilst a line of standing vehicles close together will render one lane of the road useless, more widely spaced standing vehicles will have a less drastic effect in some cases.

The time required to pull out of line to pass a standing vehicle is practically independent of speed, and is of the order of 6 seconds. The distance covered while performing the operation is then $6v$ feet, where v denotes the speed, in feet per second; if the standing vehicles are closer together than this, the speed of a car must be reduced accordingly if it is to be brought into the near side of the road between them.

If standing vehicles reduce the road to a single lane, and it is used for traffic both ways, the capacity may be estimated by treating it as a controlled bottleneck. Then the capacity

$$= \left(1 - \frac{a + r}{b + r}\right) S \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

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[1] *Loc. cit.*

[2] F. Garwood, Journal of the Royal Statistical Soc., vol. 1 (1940–41), p. 65, January 1940.

Substituting suitable values, the capacity becomes

$$\left(1 - \frac{12 + 6}{30 + 6}\right)S = \tfrac{1}{2}S;$$

that is, one-quarter of the capacity of an unobstructed two-lane road.

The foregoing observations may assist in solving some problems, such as estimation of the improvement to be gained by imposing restrictions of some kind or other upon waiting by vehicles.

Conclusion.

The methods described can be applied to the solution of specific problems in traffic-control and the design of street improvements, or to the determination of general principles. Their applications are, however, too numerous to detail in a short Paper. Considerable scope exists for further investigation into quantitative methods of traffic study.

Acknowledgement.

The Author wishes to acknowledge assistance given by Mr. W. F. Adams.

The Paper is accompanied by three sheets of drawings, from which the Figures in the text have been prepared.

APPENDIX.

Delay to Vehicles at a Stoppage of Known Duration.

c denotes total cycle.
s ,, stop period.
l ,, loss of time in accelerating.
a ,, time of arrival at stop after commencement of stop-period if unobstructed.
d ,, average delay to delayed vehicles.
n ,, number of vehicles delayed per lane.
S ,, saturation density.
D ,, density of traffic on road considered.

Considering the conditions in one lane (*see Fig. 4*) :—

$$\text{Delay to vehicle A} = s + l + \frac{3{,}600}{S} - a_a$$

$$\text{,, \quad ,, \quad B} = s + l + 1 \times \frac{3{,}600}{S} - a_b$$

$$\text{,, \quad ,, \quad C} = s + l + 2 \times \frac{3{,}600}{S} - a_c$$

$$\text{,, \quad R} = s + l + (r - 1)\frac{3{,}600}{S} - a_r$$

Delay to last vehicle

$$= s + l + (n - 1)\frac{3{,}600}{S} - a_n \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$d = s + l + \frac{(n-1)}{2}\frac{3{,}600}{S} - \frac{a_n}{2}$$

But

$$a_n = s + l + (n - 1)\frac{3{,}600}{S} - \frac{l}{2} \quad . \quad . \quad . \quad (2)$$

from (1) above, using the approximate value of $\frac{l}{2}$ for the mean delay to the last vehicle,

$$d = s + l + \frac{(n-1)}{2}\frac{3{,}600}{S} - \tfrac{1}{2}\left\{s + l + (n-1)\frac{3{,}600}{S} - \frac{l}{2}\right\}$$

therefore $d = \frac{s}{2} + \frac{3l}{4}$ . . . . (3)

Also, approximately, $a_n = \frac{3{,}600n}{D}$;

from (2),

$$\frac{3{,}600n}{D} = s + l + (n-1)\frac{3{,}600}{S} - \frac{l}{2}$$

therefore

$$\left(\frac{3{,}600}{D} - \frac{3{,}600}{S}\right)n = s + \frac{l}{2} - \frac{3{,}600}{S}$$

$$n = \frac{s + \frac{l}{2} - \frac{3{,}600}{S}}{\frac{3{,}600}{D} - \frac{3{,}600}{S}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The mean number of vehicles per lane per cycle

$$= \frac{Dc}{3{,}600}$$

average proportion delayed

$$= \frac{3{,}600n}{Dc}$$

$$= \frac{3{,}600}{Dc} \times \frac{s + \frac{l}{2} - \frac{3{,}600}{S}}{\frac{3{,}600}{D} - \frac{3{,}600}{S}}$$

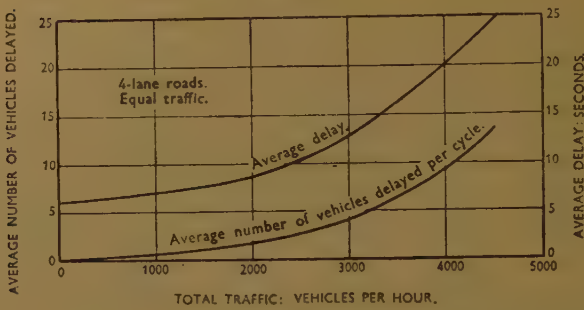
$$= \frac{s + \frac{l}{2} - \frac{3{,}600}{S}}{c\left(1 - \frac{D}{S}\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

The average delay to all vehicles in the stream considered is the proportion delayed times the average delay to those delayed.

$$= \frac{\left(\frac{s}{2} + \frac{3l}{4}\right)\left(s + \frac{l}{2} - \frac{3{,}600}{S}\right)}{c\left(1 - \frac{D}{S}\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The relation of vehicles delayed and the average delay to the volume of traffic for a typical case of fixed-time signals is shown in *Fig. 5.*

Fig. 5.



The device of using $\frac{l}{2}$ for the mean delay to the last vehicle is only justified by the fact that for normal values of the other quantities it is approximately true and its use gives practical expressions for d and n. For values of D below 200 the simpler method should be used.

Discussion.

The Chairman emphasized the difficulty of arranging meetings of the Section under war conditions, and stated that an effort had been made to meet at least once a year. He hoped that it would be possible to meet more frequently; and the Council would welcome the submission of Papers for discussion.

The Author observed that the Paper dealt with the theory of road traffic flow, and therefore did not contain many practical suggestions. Owing to the war it could not be supported, as it should have been, by adequate new data. He also wished to apologize for any clumsiness in the manner in which the mathematical arguments in the Paper had been set out.

With regard to the taking of statistics, he considered that a quite short traffic census could be adequate. From that point of view, the denser the traffic the shorter the count could be; for example, in a street such as Oxford Street, a count of about 10 minutes on each stream would be sufficient, provided that it were started at one release of the traffic by a signal or police control and finished at the next release. Generally speaking, he thought that thirty vehicles in any significant stream would probably be sufficient for a figure indicative of the behaviour of the traffic to be obtained.

With regard to the history of the subject, Colonel O'Gorman had written a Paper, he believed, just before the last war, in which

suggestions were made upon which the early work in 1924 was based. At about the same time the Board of Trade London Traffic Section had endeavoured to produce a scale of values for various types of vehicles, with the idea of formulating a Board of Trade unit of traffic congestion. The idea was that an ordinary car should be regarded as having the value 1 unit, an omnibus 15 units, a tram 20 units, and so on; but unfortunately the method was found to be impracticable, and had to be abandoned.

The Paper dealt with the problem of density per traffic-lane, and the few figures given were taken merely for the purpose of illustration, rather than as working figures. He could cite other cases that might be of interest. For example, the "Angel", at Islington, showed a total of 1,200 vehicles per hour per traffic-lane. The Holland vehicular two-lane tunnel under the Hudson river, New York, when running at full capacity, showed a density of 1,250 vehicles per hour per traffic-lane; its entrances, of course, allowed the full capacity of the tunnel to be utilized. The Haymarket at the north end, at a time when the road was obstructed by repair work, showed a density of 1,800 vehicles per hour. It would be noticed that all of those figures were of the same order of magnitude. A high degree of accuracy could not be expected, perhaps, but the figures seemed to agree with the proposition advanced in the Paper, that a working figure would be between about 1,200 and 1,800 vehicles per hour, according to the type of traffic.

An interesting paradox in traffic was visible in the effect of the bottle-neck in a road carrying traffic in one direction, for example, leaving a football-ground, it was found that the traffic in the one-lane section moved at about 20 miles per hour, whilst the traffic in the two-lane section, before it reached the obstruction, moved at only about 4 miles per hour. That seemed rather strange, but, examination of curve B in *Fig. 1*, p. 252, *ante*, would show that half the maximum capacity was reached at a speed of about 4 miles per hour. Therefore, if more traffic tried to get through the bottle-neck than could be carried by it two lanes were formed, with a volume of about 900 vehicles per hour in each lane, which corresponded with a speed of 4 miles per hour. He suggested that it was possibly that effect, combined with others, which resulted in the slow traffic in the streets with very dense traffic; that was to say, refuges tended to canalize the traffic into dual carriageways, and an omnibus-stop narrowed the two lanes into one; the traffic passed the omnibus at the stop at a good speed and dribbled up behind at about walking-pace.

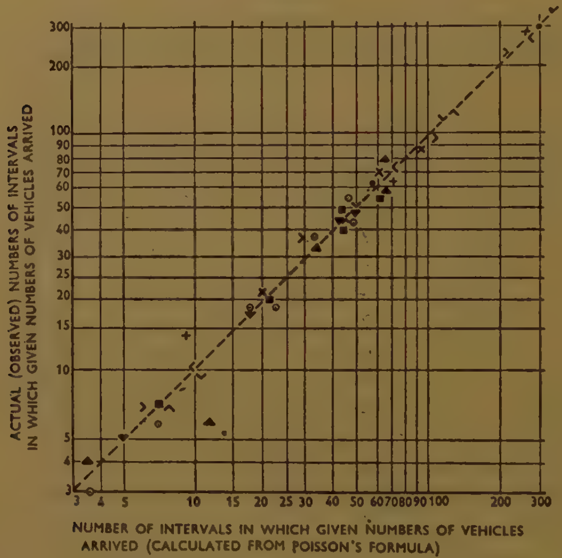
With regard to the effect of slippery roads, the figures used had been taken from the Paper by Messrs. Batson, Bird, and Stradling[1] because they happened to be available; probably they were not strictly applicable to skidding in the forward direction; but he considered that they provided sufficient indication of the effect of a slippery road, which, it would be seen,

[1] *Loc. cit.*

reduced the economic speed to about one-half, and also lessened the capacity of the road.

In the past the cycle formula had probably been the most frequently used formula for various purposes. One of its first uses was in fixing the relation between the traffic volume and the cycle to be adopted for traffic-

Fig. 6.

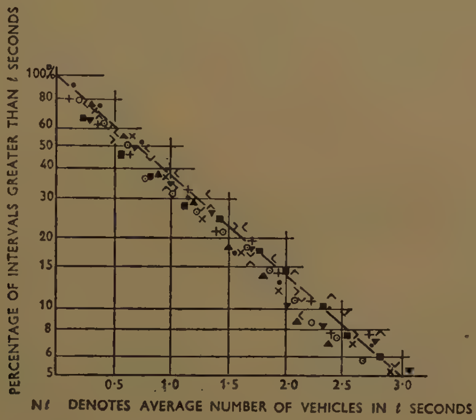


Symbol.	Site.	Traffic vehicles per hour.
●	Whitehall	744
■	Cromwell Road	512
▲	Sloane Street	362
▼	Royal Hospital Road	306
×	Holles Street	234
>	High Street, Canterbury	189
∨	Bothwell Street, Glasgow	152
∧	Sidcup Road	128
<	Footscray Road (Northbound)	95
+	New Dover Road, Canterbury	93
•	Footscray Road (Southbound)	70

signal installations in which the cycle-time was varied according to the actual volume of traffic, instead of being varied in the normal manner, by the incidence of arrival of the traffic; that was to say, if a mechanism had to be designed for varying the cycle according to the number of vehicles which had passed during the previous 5 minutes, some relation must be established between the cycle-time and the volume of traffic; and the

cycle formula had been actually used for that purpose. Many traffic signals had to be adjusted by people who had not much experience, and he suggested that the first trial should be made by using the cycle formula; it might be wrong, but it was more likely to be right than was guesswork, by which, in his experience, the setting was usually much too high. He had been particularly interested lately in the use of calculations for road

Fig. 7.



Symbol.	Site.	Intervals.	Traffic vehicles per hour.
◉	Whitehall	1 second	744
■	Cromwell Road	2 seconds	512
▲	Sloane Street	3 ,,	362
▼	Royal Hospital Road	4 ,,	306
×	Holles Street	5 ,,	234
>	High Street, Canterbury	10 ,,	189
V	Bothwell Street, Glasgow	10 ,,	152
Λ	Sidcup Road	10 ,,	128
<	Footscray Road (Northbound)	10 ,,	95
+	New Dover Road, Canterbury	10 ,,	93
•	Footscray Road (Southbound)	20 ,,	70

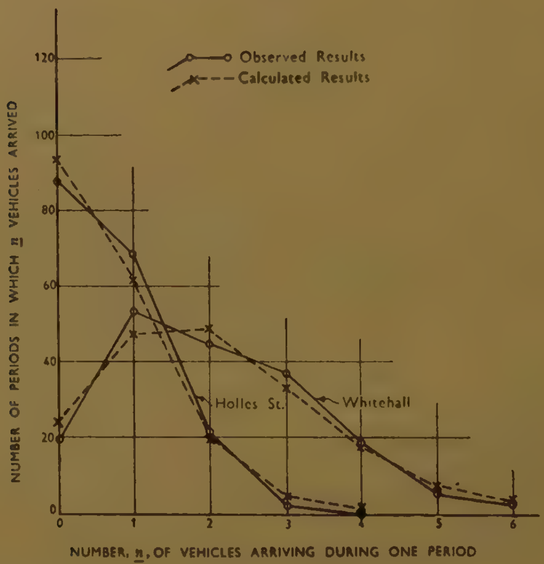
improvements, wherein a knowledge of the effect of some particular improvement was required; for example, the widening of a road, as compared with a roundabout. At least such calculations furnished an indication. Experience could not be relied upon entirely, because no two cases were alike.

He wished to mention another form of calculation which was not referred to in the Paper, namely, the preparation of a revised traffic working diagram. The best assumption to adopt was that vehicles would proceed to their final destination by the shortest route which the regulations permitted them to take. Sometimes, however, traffic was diverted so far

from its route that drivers were disinclined to return to it. He thought that the best way to establish the diagram was to plot the traffic arriving at all the places where it entered undisturbed and then estimate the volume of traffic going through at a particular point in the circumference of the roundabout. In the case of Trafalgar Square, eleven schemes had been prepared before that finally put into operation was decided upon. He believed that in Paris such schemes used to be tried out with a large party of mounted police or soldiers, but he thought it was better for the temper of the people to experiment on paper rather than on the site.

Although the Paper was not at all topical, it had some topical applications. It was possible to make some estimate of the delay which would

Fig. 8.



be experienced at a bomb-crater where room was available for one lane of traffic to pass. He had not yet prepared Tables for the purpose, but they could be prepared, and would be worth while. It would then be possible, knowing approximately the volume of traffic along the road, and also the distance entailed by a diversion, to determine whether it would be better to adopt the shuttle system or to divert one stream right round. The question often arose whether it was justifiable to delay the completion of the whole job in order to keep the road open for one lane of traffic, and it might be worth while to have a Table to which reference could be made. The actual results would probably lie well on one side or the other; if they came on the border-line it would not matter which system was adopted.

The Author exhibited three lantern-slides (*Figs. 6, 7,* and *8*) illustrating the random distribution to which he had referred very briefly in the Paper. He drew attention to the reasonably close agreement between the observed results and those predicted, and pointed to the fact that although the observations were taken in different parts of the country, and on different types of road, the agreement was not affected.

He observed that when he began to write the Paper he thought that he knew a little about traffic calculations; but when he had finished it he realized that his knowledge was far less than he had imagined. Somebody who had studied traffic theory closely had remarked, in desperation, when dealing with one particular problem, that the population of the United States of America was 120 millions, and there were 120 million traffic experts: and the Author thought that, with an adjustment of the figures, a similar statement might be made about Great Britain.

Colonel Mervyn O'Gorman observed that so long ago as 1926 he had had the idea that a law of flow for motor traffic might, even if only approximate, help to improve traffic control, to augment the economic use of roads, and to reduce road accidents. He considered that analogies from hydraulic flow were misleading. It was obvious that a head of pressure, that was, an accumulation of many urgently-driven vehicles behind the unit considered, produced no effect that could be likened to a pressure to move it forward more quickly. At any time on the Portsmouth road it could be observed that when vehicles were travelling as a discontinuous stream the speed of any individual in that stream was, in a vague manner, decided by the length of the vacant gap in front, that was, the "interspace" between it and the next vehicle ahead. That rather trifling observation was novel on the 27th April, 1927, when Colonel O'Gorman published it in the *Times* and sent the elementary algebra relating to it to the Ministry of Transport, demonstrating also that that type of attack must have some value by posing and answering the following questions:

(1) When will vehicles in a single traffic lane provide the maximum of flow?
Answer. When they are one vehicle-length apart.

(2) What speed, in miles per hour, will give this maximum of flow?
Answer. About 14 miles per hour.

(3) How many vehicles will pass per minute?
Answer. About 30 per minute, or 1,800 per hour.

(4) Which would be more disorganizing to traffic output, a lower speed than the best or an excess of speed?
Answer. Driving too slowly by 7 miles per hour is worse than driving too fast by a similar amount.

The following was a word diagram. One taxi-cab, with orders to circulate at 30 miles per hour, would be put on a road forming a closed circle 440 yards in circumference. The number of cabs would then be

increased to five, ten, twenty, and so on up to 110. When those vehicles were nose to tail they would hardly be able to proceed at 4 miles per hour. In fact, as the vacant space in front of each car became smaller its speed would become less. At some intermediate number of cars—at a guess, fifty-five—the speed would be about 20 miles per hour if the drivers were careful to space themselves equally.

That guess was uninteresting until it was realized that a half-loading of fifty-five cars travelling at 20 miles per hour represented 250 per cent. more traffic-flow than did 110 cars at 4 miles per hour, or that $2\frac{1}{2}$ times more cars would be cleared past a given point if they were spaced, than if they were allowed to crowd in as tightly as they could, and as, in fact, they did.

To-day, 16 years later, he still did not think that the method suggested was faulty within the reasonable assumptions made; but the point he wished to make was that such calculations were capable of opening up many useful lines of thought. The Paper provided an instance of that, and the Author had utilized his equations for the timing of traffic-control lights. Colonel O'Gorman did not doubt that the Author recognized how very suggestive for other traffic problems such methods were.

The following elementary calculations indicated that the safe speed of urban transport was dominated by brake power, and not by the weight of the vehicle or by its horsepower, and that the road surface and road slope influenced the most economic rate of travel of a traffic-column. Each traffic-speed required a particular length of interspace between the units. Those considerations showed that "cutting-in" (hitherto a vague term of abuse) was sometimes noxious and sometimes permissible, depending upon whether the "cutter-in" intruded his vehicle into the interspace required by the speed of the vehicle overtaken. Justification for the driving-school dictum, "Always prefer to overtake on an uphill," could be found in the fact that interspaces for uphill travel were shorter than for the level or downhill. Similar simple reasoning showed why a long clearance should be allowed for when overtaking on a slippery road. Indication was also given why a side junction to a major road might slow the traffic so surprisingly, namely, that it shortened all the interspaces. The law of interspaces taught that a "blind bend" was not always "blind", but was blind only to those drivers who found that the obstruction intruded into the interspace that they required for the particular speed of travel they were adopting. A town with overburdened streets could relieve its traffic-density, (*a*) by inducing drivers to avoid driving nose to tail; (*b*) by keeping the streets clean and not slippery; (*c*) by using roundabouts exclusively as non-stop vortexes at the speed of maximum output. Moreover, since the assumptions made were very rough, it was evident that a research body on traffic-flow and accidents, competent to measure accurately those data, and also such other data as small increments of traffic-fluidity in relation to control, could firmly

establish the principles upon which congestion and its attendant accidents could be reduced considerably. The saving would then be measured in millions of pounds.

If v denote the speed of the car in feet per second, f the acceleration (negative) produced on a car by braking, x the space, in feet, in which the cars came to a stop, from the time when the driver intended to apply the brakes, and L the length, in feet, of the vehicle (average) plus a varnish clearance from its neighbour ahead, then, since $v^2 = 2fx$, $x = \frac{v^2}{2f}$, and when, in a moving line of vehicles, the distance from the nose of one car to the nose of the next was $x + L$, that distance $= \frac{v^2}{2f} + L$. The number of vehicles to pass a given point per second was $\frac{v}{x+L}$, which equalled $\frac{1}{\frac{v}{2f}+\frac{L}{v}}$.

That expression was a maximum when $\frac{v}{2f} = \frac{L}{v}$, which reduced to $v^2 = 2fL$. Comparing that expression with the previous one, $v^2 = 2fx$, it was found that for maximum flow $L = x$, that was, the vehicles should be separated from one another longitudinally by a distance or interspace equal to their individual length. That was not quite correct, because, of course, deceleration on braking did not start from the moment when the driver intended to put his brake on; he had to have his mental reactions, and the time when he intended to put his brake on differed from the time when he actually put it on. Nevertheless, the formula $L = x$, would give approximately the maximum number of vehicles along a single traffic lane.

Assuming $L = 20$ feet; and f (a high rate of braking) to be 10 feet per second per second. Then $v^2 = 2fx$ became $v^2 = 2 \times 10 \times 20 = 400$, and $v = 20$ feet per second. Therefore $V = 14$ miles per hour. At that rate of braking, therefore, 14 miles per hour was the ideal speed for the maximum flow at which the maximum flow of vehicles per hour could be realized over some bridge, tunnel, or other similar section of roadway without side entrances.

The number of vehicles passing per second was $\frac{v}{x + L} = \frac{20}{20 + 20}$ per second $= \frac{1}{2}$ vehicle per second, that was, 30 per minute, or 1,800 per hour, per traffic-lane.

If the speed were halved to 7 miles per hour, or 10 feet per second, that was, if the interspace were reduced, the number of vehicles would be

$$\frac{10}{\frac{100}{20} + 20} = \frac{10}{25} = 0{\cdot}4 \text{ vehicle per second.}$$

Therefore the output or carrying-power of the bridge would be reduced by

20 per cent. As was well known, a greater reduction in flow occurred when the vehicles were driven in the manner affected by drivers to-day, so as to have only a negligible clearance; that would reduce the speed to 3 or 5 miles per hour.

If the speed were increased to 20 miles per hour, the number of vehicles passing would again be less than the number at 14 miles per hour, namely, about 9 per cent. less (accurately 0·46 vehicle per second). Thus there was less disadvantage to flow if the interspaces were increased by some small measure than if the interspaces were decreased by an equal measure. That conclusion had some practical value, because if traffic-controllers were ever to require drivers to observe an interspace, it would be known that to exaggerate the interspace (a likely fault with an inexpert driver) was not so pernicious to flow as was the taxi-drivers' fault of cutting interspaces much too fine.

Colonel O'Gorman observed that the practicality of the examples was the more remarkable because the approximations were admittedly very rough. The ability of the driver to stop was considered, but his even more valuable ability to avoid collisions by steering to clear the obstacle was disregarded. The variable coefficient of friction of various road surfaces was assumed to be the same for all vehicles and all speeds, etc. Tributary roads were assumed to be non-existent.

Captain F. A. Rayfield wished to emphasize the fact that the Paper dealt with a subject about which far too little had been heard in the past. There had been too much loose talk, particularly about the hydraulic analogy, which was not strictly true, and the sooner quantitative analysis was undertaken the better. About 10 years ago he had made some investigations into traffic capacities and economic speeds, and, although his records were not available, he felt sure that the results corresponded very closely with those given in *Fig. 1*. He had been surprised at the time, because he had not seen the statements put forward before, and he had doubted his own results; but careful checking had led him to the same conclusion.

He was also particularly interested in the analysis of the methods of ascertaining the capacity of intersections. He was quite sure that the traffic problem could be solved only by attention being paid, in the first place, to intersections. In great cities in the past considerable money and time had been spent in widening streets, but little or no attention had been devoted to dealing with the additional load placed upon the intersections by that method of procedure. Even in London one could think of many cases where the widening of a street had increased the traffic-delays. In short, if a given sum of money were available for solving the traffic problem (there was never enough for that purpose, and he supposed there would not be enough in the future), greater value could be obtained by spending it on intersections than in any other way.

The quantitative approach to the capacity of intersections was therefore

doubly important, and it enabled alternatives to be placed before those responsible for the expenditure of public money. It was possible to say, for example, that three different methods of solving a problem would give certain degrees of improvement in the existing conditions, and that those various methods would cost £X, £Y, and £Z respectively; and it was for authority with financial responsibility to say which method should be adopted.

A fairly recent example in London was that of the treatment of an important bridge-head. Two methods were available by which the traffic delays anticipated at the northern approach could be reduced. One was a relatively simple and inexpensive method which would have resulted in an improvement in traffic-capacity and a reduction in traffic-delays which, when integrated, would have effected an improvement of 30 per cent. The other was a very comprehensive improvement, costing twelve times as much, which would have resulted in an improvement of about 70 per cent. It was a matter for consideration whether the additional improvement would justify the higher expenditure; and it was only by such methods as were outlined in the Paper that scientific approach could be made.

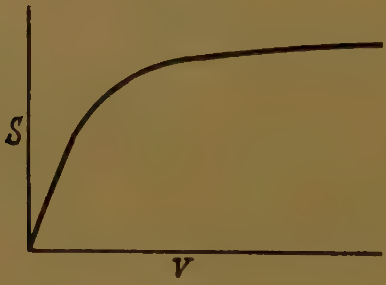
Although some of the suppositions and the bases for the calculation of capacities might not be strictly accurate, so long as they were the same in each case a comparable basis could be obtained; and the lantern-slides which had been shown indicated that they were often very close to the actual mark in practice. That led to a study of intersections, and naturally one would like to start any comprehensive scheme of improvement by dealing with those intersections at which traffic-delays, and therefore the cost to the community, were the greatest. If that were done in London, for instance, one found that it was not at such important and heavily-trafficked intersections as Hyde Park Corner that the community suffered most. He thought he was right in saying that Ludgate Circus and certain places in south London, such as the "Bricklayers' Arms", in Old Kent Road, would yield a much higher return for money spent on their improvement than would some of the better-known intersections.

Although it might not be strictly relevant to the subject of the Paper, he wished to express the hope that architects and others who prepared town-planning schemes might have their attention called to the necessity for a specialized study of traffic. Town-planning consisted in the main of two principal sub-divisions: zoning, which was largely a matter for architects, and transport, which, he submitted, was a matter for engineers with road experience. He hoped, therefore, that the Ministry of Transport road engineers, and those who dealt with roads in ordinary life, would be allowed to play their part in any scheme of reconstruction.

Mr. W. F. Adams observed that at any given speed of traffic the maximum volume that a single lane could carry could be expressed by the equation: $S = \frac{5280V}{h}$, where V denoted speed, in miles per hour, and h

the distance, in feet, between heads of vehicles. Then, either h depended upon the traffic-speed, or it did not. If it did not, one could give up the idea of a "traffic science"; but if h did depend upon the traffic-speed, the equation could be written $S = \frac{5280V}{f(V)}$. Colonel O'Gorman had regarded $f(V)$ as a quadratic, the Author had regarded it as a linear function for cyclists and a quadratic for other traffic, and other people had made other suggestions. The Massachusetts Highway Accident Investigation Board had suggested something depending upon the 2·2th power. If, for a certain speed, the total traffic had a maximum, it meant that $\frac{f(V)}{V}$, that was, $V^{-1}.f(V)$, was a minimum. Differentiating, the speed for maximum traffic could be calculated from $V^{-1}.f'(V) - V^{-2}.f(V) = 0$, which gave

Fig. 9.



$V.f'(V) = f(V)$. That was straightforward mathematics; the whole of it followed once it was conceded that the distance between vehicles must depend upon the traffic-speed.

The next step was to consider what the function might be. If $f(V)$ were linear, $f(V) = AV + B$ and $f'(V) = A$; hence $AV = AV + B$. That expression could not possibly be satisfied unless $V \to \infty$: therefore if $f(V)$ were linear there was a limit to traffic, but no saturation, and a curve of the type illustrated in *Fig. 9* was obtained. The Author had given a formula of that type for cyclists, and it would be interesting to know whether any evidence was available to support it.

The next simplest form to the linear was the square law, $h = f(V) = AV + BV^2 + C$, and a quick rise to saturation occurred, followed by a less rapid fall (*Fig. 10*). The same form applied if any power higher than the second was involved.

If $f(V) = A_0 + A_1V + A_2V^2 \ldots A_nV^n$, then the speed which produced the maximum volume of traffic could be found from

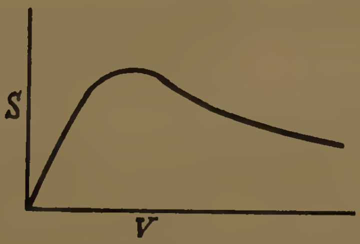
$$\sum_{r=2}^{r=n}(r-1)A_rV^r = A_0.$$

It was curious and interesting that everyone who had had to deal with traffic in mass, or to study the properties of traffic in mass, had come to the conclusion that the second type of curve was the more appropriate; yet at least half of those who had studied the actual spacings between vehicles had arrived at a curve of the first type.

The divergence between those who studied the mass of traffic and those who studied the spacing between vehicles opened up a field of research requiring no more apparatus than a pencil, a notebook, and a stop-watch, although other means had been used, for example, cine-cameras, aeroplane-cameras, and portable range-finders.

The Author had stated that, "Attempts to measure the distances between moving cars at various speeds have not yielded consistent results"; Mr. Adams agreed that that was true. The important point, he thought,

Fig. 10.



was that the type of curve rising to a maximum and falling again provided a conservative basis for traffic design. It was found from that curve that saturation traffic was about 1,500 to 1,800 vehicles per hour, and not the 3,600 or 4,800 vehicles per hour given by most authorities for the linear type of curve.

He agreed that the hydraulic analogy was misleading; and the electrical analogy for traffic could be equally misleading. Possibly less misleading than others was the kinetic theory of gases, in which a number of independent particles were considered, travelling at different speeds and moving independently of each other; but the idea of each individual vehicle suffering something like half a million collisions per second "were a thing imagination boggles at."

Under the heading "Units and Definitions" the Author had stated: "The number of 'lanes' of traffic a road can carry is the number of vehicles which can move abreast or pass upon it; in a two-way road this may be an uneven number if there are no central obstructions." Mr. Adams considered that the words "in a two-way road" could quite well be omitted.

The Author had also stated that: "The denominator of the right-hand side of equation (2) represents the headway, in feet; this is not the full pulling-up distance, as consideration will show to be reasonable, since

any cause for stopping will be visible in front of the forward car." Mr. Adams thought that an even more important reason why the denominator was not the full pulling-up distance was that the forward car could not stop instantaneously, and that the driver who was following made an unconscious allowance for that fact.

Equation (7) was very interesting, because it showed that at any intersection where control was necessary, that was, any cross-road with heavy traffic, the maximum traffic-capacity T of a simple intersection could never be greater than half the traffic-capacity of the streets entering it. It was evident that $\left(1 - \frac{2a}{c}\right)$ could not possibly be greater than 1, and MS represented the total amount of traffic that could enter the junction by all streets.

He wished to utter a word of caution with regard to the statement on p. 258 that: "The small roundabout has not always twice the capacity of a signal-controlled intersection. For example, in this case, if the roads were double the width, the signals would have double the capacity, but the capacity of the roundabout would remain unaltered." That statement might be taken as implying that it would always be possible to make a signal-controlled junction pass as much traffic as a roundabout if the roads were sufficiently wide. That, of course, was not true. The paragraph applied only to the small roundabout, that was, the roundabout which was not sufficiently large to pass all the traffic which could enter it from all the contributory streets.

With regard to traffic planning, the ill-fated Charing Cross bridge scheme might, perhaps, be cited as the first case of deliberate traffic planning in Great Britain. That bridge would have had its drawbacks, but at least it would have been capable of carrying more traffic than any other London bridge.

Mr. Adams particularly appreciated the robust practical approximations which had always characterized the Author's work.

Mr. R. B. Hounsfield observed that more than eighteen months had elapsed since his work in connexion with traffic had been brought to a stop, and his only means of keeping abreast of events had been to watch the Press. Although many references had been made to town planning and to post-war architecture, he had seen no reference to the necessity for comprehensive regional traffic planning, and he was rather disappointed to find that the Author had confined himself within the limits imposed by pre-war conditions, when only isolated junctions or possibly two or three junctions close together in a short length of street could be dealt with, and any attempt to think further afield was of merely academic interest. He thought that it would be agreed that most of the discussion on the Paper had been concerned with individual junctions, except for Captain Rayfield's remarks about the necessary relationship between traffic planning and town planning. He was glad that Colonel O'Gorman had emphasized

the importance of traffic research in war time, but since no reference had been made to the necessity for regional traffic planning, he would like to take the opportunity—rather rare in war-time—to make a plea with regard to that. He wished for definite steps to be taken to establish regional traffic planning in Great Britain, with the status that it deserved. Traffic planning should follow on automatically from town planning, at all events in large areas: in small areas it might not be necessary. At long last town planning in Great Britain was receiving the attention due to it. Its purpose was to draw up a general outline of the distribution of life in a city. In a large area it should then be ascertained where traffic volumes would exceed the capacity of the ordinary street system. It would then be possible to obtain a general idea of the traffic network required to handle that traffic, and the architect could be informed not only of the regional distribution in a city, but also of the traffic needs that had to be met. Unfortunately too many architects visualized wider and wider streets as the only solution of the traffic problem. Naturally an aversion existed to enormous traffic arteries in the centres of cities, and that was probably the main cause of a large number of suggestions for prohibitory and distortive regulations to deflect traffic and cause it to behave in an unnatural manner. Distortion of traffic was just as detrimental to the life of a city as the distortion of the breathing apparatus would be to the human body. London had a character of its own, and the provision in London of wider and wider streets to carry larger and larger volumes of traffic was altogether foreign to that character. In order that the character of London might be preserved, it was important that the traffic problem should be handled in a positive manner, and not by means of evasive and prohibitory regulations.

Mr. Hounsfield wished to put his plea for comprehensive regional traffic planning in a definite form. He realized that traffic staffs had been decimated, and that the Author, as one of the few survivors, was very busy. It was all the more important, therefore, that the limited time available should be spent in studying the broadest issues of traffic behaviour.

He suggested that investigations should be made into the relationship between the sizes of cities and the volumes of traffic to be encountered, and also between the distances travelled and the volumes of traffic travelling those distances. Further, the relative attractiveness of trunk roads and local streets for journeys of various lengths, in regions where both trunk road and local street networks existed, should be ascertained. Thus a basis for calculating the probable volumes and distribution of traffic in a post-war city would be provided.

He realized that it would be difficult to carry out some of that work in war time, but he felt that much of the difficulty could be overcome by seeking the co-operation of people abroad. For instance, considerable traffic research and regional traffic planning had been carried out by the Traffic Department of the Port Authority of New York under Mr. Cherniak,

and he felt that great benefit would be derived from a study of the results of that research work, which had been carried out over a number of years. He considered that a start made on those lines would establish regional traffic planning in Great Britain with the status necessary to ensure the correct design of our cities. It might enable some very serious mistakes to be avoided, and prevent people from grossly underestimating the magnitude of the problem.

It was important nowadays to study traffic planning and the possibilities of the future, not because bombing had facilitated the re-laying of streets from a material point of view, but because it had created such an upheaval in the public outlook that the possibilities were out of all proportion to the number of buildings destroyed.

Fig. 11.

Town planning, therefore, should be automatically followed by regional traffic planning in a large area. The traffic problem would then be properly handled and solved, and not only would a positive constructive solution be found, but also the character of our cities would be preserved.

Dr. F. Garwood said that the hydraulic analogy had been condemned by previous speakers, but he thought that, if interpreted with caution, it was useful in visualizing some of the problems and calculations with which the Author had dealt. For instance, in the calculation of equation (5), where the Author gave the formula for the general case of a multipart cycle of n phases with a dead period of r seconds, he envisaged that problem somewhat as follows.

He assumed a number of channels leading to a junction and a road feeding each channel at densities D_1, D_2, and so on all the way round (*Fig. 11*). Each of the inlets could pass the traffic at a saturation-density S, which would obviously be greater than the sum of the D's. Starting from the " go " period for one inlet, all the other inlets being closed, traffic flowed in at a density S while it was being held at each of the other points. The " go " period, g_1, had to be reduced to $g_1 - a$, the reason being, according to the Author, the constant time required to reach saturation. The traffic would run out for that time at a rate S. That would go on all

the way round until the completion of the time $g_1 + g_2 \ldots + g_n$, the sum of all the "go" periods, but the cycle was not quite complete, because there was a dead period, r, presumably to allow pedestrians to cross the road. The question was, whether that cycle-time was sufficient to prevent each of the inlets being blocked up, and he imagined that that might be a problem of some practical importance. To take a very simplified example, suppose the first channel to be 600 feet long and the traffic at 60-foot intervals, so that when it was actually flowing through freely, there would be ten vehicles in the channel. If each vehicle were effectively 15 feet long, and the traffic were held up, 150 feet of the approach would be occupied, leaving 450 feet, that was, room for thirty more vehicles. If the length of the approaches were known, it would be possible to obtain a value for each of the extra capacities. It was then possible to see what mathematical conditions the cycle must satisfy to prevent those capacities being exceeded, that was, to avoid congestion at the farther ends of the channels.

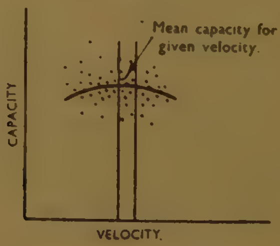
He thought that the Author's formulas (1) and (2), in relation to saturation-density and velocity, might be misleading, as tending to make people think that if the velocity were given the capacity could be calculated, or vice versa; but in practice he did not think that was so. If observations could be taken of velocity and capacity, or of distance spacing, it would be very surprising to find always that one quantity was related to the other. He thought there would always be a tremendous variation, and it was necessary to go a little more carefully into the meaning of the equation in question before applying it in practice and before drawing any conclusions; it was important to define exactly what was meant by "velocity" and "capacity." He agreed that in a street in which traffic was released very freely it usually moved in a more or less solid homogeneous flow, and therefore the velocity must be much the same for every vehicle; but, as the Author had pointed out, the spacing would not be consistent, and Dr. Garwood did not think that was surprising. Suppose observations of velocity were taken at a given point, the capacity measured by counting the vehicles passing in a fixed time, say 1 minute, and the capacity plotted against velocity, then a scatter of points would be apparent. The longer the period over which the spacing was taken the smaller would be the scattering.

He could not say what connexion that scatter diagram, which was a representation of the real facts, had with the curve of the equation referring to saturation-density and velocity which was a theoretical equation based on some hypothesis. One could reduce the scatter diagram to a curve by considering the arrays of points corresponding to short elements of velocity, as in *Fig. 12.* The mean of each array would follow a curve which could be interpreted as the average capacity attained by vehicles actually travelling at a given velocity. Clearly that was not the maximum capacity, since probably half the points of the diagram would be above the curve.

The relation of that curve to any theoretical equation was a matter for further investigation. He realized the difficulty of taking observations. Conditions were not the same as they were on highways, rural roads, and by-passes. On such roads in America observations had been carried out and curves had been calculated, which were effectively estimated from a scatter diagram.

He considered that *Fig. 4* was an interesting and instructive representation of the phenomena at a traffic junction. He did not quite follow the reason for using the expression $\frac{l}{2}$ for the mean delay to the last vehicle delayed; he thought that it would be adequate to assume that the delay to the nth vehicle was the minimum, and that the next vehicle just got through without delay, so that it followed on the preceding one after a spacing of $1/S$, the saturation-spacing. However, the result he had

Fig. 12.



obtained on that basis was near enough to the Author's formula (6) for practical purposes.

Mr. N. Herwald considered that the speed of vehicles was determined by road rather than the width of the road by the speed. He could not see the use of the intersection-factor. In the case of Whitehall the Author had divided 550 (presumably the number of vehicles per hour per traffic-lane) by the intersection-factor and had asserted that the road was clearly working to the limit of available capacity. Therefore 550 was the saturation-density, and Mr. Herwald did not understand why the Author had divided it by the intersection-factor and produced a figure of 1,500. The figure of 1,500 was not stated as any particular unit, and he assumed, therefore, that that figure, and not 550, was the saturation-density.

With regard to headway, Mr. Herwald could not see that the length of a car bore any relation to it. The headway had been expressed in time-units, but he did not see why the time taken to travel the distance between the cars should bear any relation to the length of the cars. With regard to the statement that, "the headway required is therefore $1 + \frac{L}{v}$ seconds," he assumed that $\frac{L}{v}$ represented the time required

for the car to pass out of its own length on the road; but perhaps what the Author meant by L was the length of the car, plus the distance between the cars, although Mr. Herwald thought that the length of the car should not be included in the headway.

Mr. Herwald took exception to the Author's statement that, "the phase should end as soon as the density falls off to any extent." The traffic-control policeman brought about that end; he waited until no more traffic was left in a road and then barred that road, to give the right of way to the traffic on the other road.

In December, 1937, in an article entitled "Some Theoretical Aspects of the Control of Road Traffic [1]", Mr. Herwald had prepared a graph in which the ordinate was the delay, in vehicle-seconds, and the abscissa was the velocity, in feet per second. The curves, which were not complete in effect because they bent down to zero, were very similar to that drawn by Mr. Adams (*Fig. 10*), although not related to the same ordinates. The curves showed, in effect, that the delay, in vehicle-seconds, increased as the vehicle stood still. When vehicles approached a cross-road against the light, it might not have changed by the time they arrived, so that they had to stop. It could not be asserted that, because those vehicles had to stop, the junction was not working efficiently; it might be working perfectly efficiently simply because vehicles were passing through the intersection. Two opposing traffic streams could not pass through the intersection at once; therefore one stream had to be stopped and the other allowed to go through. It had been found by experience that short cycle-times (that was the change from one road to the other and back again) were better than long cycle-times, because of the fact that vehicles were stopped by the red light, and the longer they stopped the longer was the delay, in vehicle-seconds. Therefore he thought that the Author's statement that, "the phase should end as soon as the density falls off to any extent", was incorrect because it assumed that there was more or less continuous traffic at the intersection; therefore only one cycle-time at that density would give efficient working of the junction, and that was not nearly so high as the actual time that the saturation fell off.

Cycle-time was not a fixed period. The duration of the old fixed cycle was unvarying, whether the traffic was there or not; but progress had been made since then, and in the case of vehicle-actuated systems the vehicles themselves set the cycle-time. If one road had no vehicles in it the signals would not change, whilst if there was one vehicle in one road and a long stream of traffic in the other road the cycle-time would depend upon the fixed settings on that road, which was there as a limiting device only and not to set the cycle-time.

Mr. W. F. Adams, referring to the tests mentioned in the Paper on velocity against spacing, observed that in tests made with Mr. D. A. de C.

[1] Journal of Scientific Instruments, vol. 14, p. 396; December 1937.

Bellamy he had obtained a result similar to that suggested by Dr. Garwood, but with one important difference. He obtained a statistical cloud of points very thin at the top and coming down more and more densely to a fairly well defined edge, which clearly indicated the minimum distance that the particular driver could maintain at all the given speeds. The Author had referred to an inconsistency between different drivers; for any one driver there seemed to be a fairly well marked edge. With regard to the cloud of points that Dr. Garwood had mentioned, Mr. Adams thought that it would be fairly thin lower down and would run up to a dense distribution.

The Chairman observed that a good deal had been said during the discussion about the analogy between the laws of traffic flow and those of hydraulics, and one was attracted to that analogy when one set out to study the subject. He had been attracted also by Colonel O'Gorman's reference to taxicab-drivers, because he supposed that such drivers were the most level set of drivers in the country, and if one could rely upon all drivers having the same skill as taxicab-drivers the analogy might be more complete. In dealing with the matter of traffic-flow, however, one must take into consideration not only the varying skill and psychology of the drivers of vehicles, but also the difference in the performance of the vehicles. He was very sensible of the contribution which had been made to the study of traffic-flow by the engineers, past and present (of whom the Author was one), in the service of the Ministry of Transport. He sincerely hoped that when the war was over the engineers of the Ministry would be given a much greater opportunity than they had at present of making known their own knowledge and experience of the subject, and that their conclusions would be accepted by those in authority.

*** **Mr. H. Alker Tripp** observed that he had read the Paper with much interest. From the practical point of view, however, he was doubtful as to the value of calculations of that character, because the computations were based (*a*) upon factors which were not constant, and (*b*) upon assumptions. In his view, when so many assumptions were needful, it would be simpler to work from actual census results and by continuous experiment.

There was, of course, the aspect of future development and the layout of new roads, but there again the same weakness was bound to be present, because the type of development, and the consequent traffic load upon any section, was so problematical. An aggregation of industrial undertakings might result in a local preponderance of heavy lorries and of tidal loads of pedal cyclists, whereas their absence would mean a faster flow of traffic, lighter in the aggregate and less tidal in character. That, of course, was elementary, but the permutations and combinations were so endless that he felt more and more driven to basing computations upon ascertained fact. In more propitious times, a considerable increase in traffic surveys would be

*** This contribution was submitted in writing.

valuable, for example, to ascertain the precise traffic-density which would cause a roundabout of given shape, size, and road-width to jam; but he was afraid that that would not be practical politics at the present time.

The Author, in reply, said that he had deliberately confined his Paper to the calculations of traffic-flow principally at junctions, because the junction was the most important feature when traffic-flow was under consideration. He had not dealt with the flow of traffic through large areas, because he considered that that was an entirely separate subject.

In the matter of traffic planning, there was, unfortunately, a tendency for a number of the 45 million traffic experts in Great Britain to be included among those who had to deal with planning. It was regarded as one of the things about which everybody knew something, and therefore the planners considered that they could deal with it in addition to all the other multitudinous things they had to consider in town and regional planning. He had lectured to the students of one of the schools on traffic matters, but he thought that most of the schools regarded traffic planning as common knowledge. He considered, however, that engineers should ask that engineering principles should be applied to both road and rail traffic aspects of planning.

Recently he had been asked to advise a student town planner, and had been very interested to see the line that the student took. On the question of the flow of traffic through and round towns he assumed that if he put a route past the town that he was laying out the traffic would go past. The Author had explained to the student that if one wanted traffic to go past a town one had either to make prohibitory regulations, which were undesirable, or to make the route past the town not only better than the route through the town but also clearly better; and, secondly, that very little traffic was through traffic. In a survey which he had carried out many years ago, he had found that only 10 per cent. of the traffic passing through the most congested part of Bond Street went from one end of the street to the other, and he thought similar conditions would be found in the cases of Oxford Street, the Strand, and other similar streets.

He welcomed Colonel O'Gorman's contribution to the discussion, because he considered Colonel O'Gorman might be regarded as the father of scientific traffic study. He did not agree, however, with the suggestion that the most economical spacing was one equal to the length of the vehicle. He thought that it would be found that the speed would be so low at that spacing that the number of vehicles that could pass a given point would be smaller than the number that would pass it at a greater spacing and a higher speed.

The density formula was rather of academic or general interest than of practical application to specific problems: that was why he had given it after his short dissertation on the fact that the number of vehicles per traffic-lane varied with something other than the type of traffic.

In reply to Mr. Herwald's question about the figure of 550 vehicles per

hour per traffic-lane entering Trafalgar Square from Whitehall, the Author had endeavoured to show that, whilst 550 vehicles per hour per traffic-lane entered Trafalgar Square from Whitehall, and not very far away, Broad Sanctuary carried practically the same number of vehicles of a similar type per hour per traffic-lane, but much more "congestion" appeared at Whitehall than in Broad Sanctuary. Two points arose in that connexion, namely, the intersection factor and the normal saturation density, which, for the type of traffic in question, as observed in Trafalgar Square, was 1,500 vehicles per hour per traffic-lane.

As a statistician Dr. Garwood had been able to help engineers in solving problems which were beyond the mathematical capacity of most engineers. The Author's observations on the Portsmouth road included the number of vehicles in different periods of time, determined by the police control. The number of vehicles in a bunch was plotted against the period. A definite tendency was evident for the number of vehicles to vary with the time, and he felt that it was justifiable to assume some consistency in the spacing of the vehicles, although the space between one vehicle and the next varied, he believed, purely in accordance with the opinion of a driver as to how near to the vehicle ahead he could safely remain.

With regard to the two alternative bases for the values of h in Mr. Adams' formula, it seemed to the Author that when a steam of traffic was flowing without much chance of it being disturbed by other factors, such as pedestrians stepping off the pavement, signals suddenly appearing, omnibus-stops, etc., such conditions as obtained on an arterial road on a Sunday night in peace time, drivers tended to follow the straight-line law, that was to say they tended to stay at such a position behind the vehicles in front that if one braked the driver behind could also brake without colliding. In towns, on the other hand, the tendency was rather for the distance to be slightly less than the braking-distance, which, of course, involved the first plus the second power of the speed. Very few opportunities existed for observing the relation between speed and traffic-density at speeds much in excess of 20 miles per hour, but on the Croydon by-pass he had observed 3,600 vehicles per hour in short bursts, and that number was very high, the spacing being 1 second.

He used the term "headway", in feet, to mean the distance from the nose of one vehicle to the nose of the next ahead, and in seconds, the time taken to travel that distance. Therefore the headway was made up of the length of the vehicle, plus a small margin so that the vehicles did not actually touch one another, plus whatever space might be needed in between.

ORDINARY MEETING.

29 April, 1941.

Sir LEOPOLD HALLIDAY SAVILE, K.C.B., President, in the Chair.

It was resolved :—That Messrs. A. S. Grunspan, J. A. Johnston, R. W. Mountain, B. D. Richards, and P. J. H. Unna be appointed to act as Scrutineers, in accordance with the By-laws, of the ballot for the election of the Council for the year 1941–42.

The Council reported that they had recently transferred to the class of

Members.

Frederick Alfred Greaves, B.Sc. (Eng.) (*Lond.*).
Ernest George Horton, B.Sc. (Eng.) (*Lond.*).

And had admitted as

Students.

Ernest John Alderton.
Gordon Philip Allen.
David Ronald Bailey.
Henry Wise Baker, Jun.
William Gilbert Bald, B.Sc. (*St. Andrews*).
Dennis Drewery Bamber.
Kenneth Thomas Bass.
Arthur Lawrence Hornibrook Baylis.
Duncan MacGillwray Beaton.
Raymond Bertram Beeson.
Derek Porteous Black.
William Mitchell Black.
Cecil Clive Bradnam.
John William Bradshaw.
Peter Hilary Cadman.
Robert Orkney Calder.
George Norris Cale.
Denis Ewing Chaplin.
Dudley James Clapham.
Frank Norman Clarke.
Nigel Hugh Clipstone.
Andrew Victor Cox.
Daniel Clark Crawford.
Alan Douglas Davenport.
John Creagh de Glanville.
Francis Dewar.
Patrick Dunlop Donald.
Robert Charles Foster.
Christopher Frank Fowler.
Nathan Friedmann, B.Sc. (*Bristol*).
John Francis Gogarty.
Thomas Eric Hepple.
Alexander Joseph Charles Hill.
Gerald Christopher John Hodgson.
Fergus Isherwood.
Alun Samuel Jones.
Eruch Jamshed Kambata, B.E. (*Bombay*).
David James Gordon Kay.
Lewis Andrew Hodgkinson Lack.
John Brown Lambie.
Michael William Leonard.
Owen Harries Lewis.
Norman Alastair Smart Locke.
Wilfred George Lockett.
Hugh Samuel MacDonald.
James Fraser McDonald.
Donald MacDuff.
John Charles Middlehurst.
Walter Millar.
James Richard Frederick Moss, B.A. (*Cantab.*).
Murdo Murchison.
Alastair Slora Ross Mutch.
Malcolm Raymond Norris.
George Donald Oates.
Peter Eames O'Flanagan.
Robert Gordon Marshall Orr.
Wilfred Joseph Oxberry.
Douglas Petrie.
Donald William Robertson, B.A. (*Cantab.*).
Peter James Scammell.
Leo Schenker.
Robert Soutar.
Jack Eric Thomson.
Magnus William Todd.
Geoffrey Whitten Tolley.
Duncan Charles Tucker.
Pierre Raymond Weigall.
Alan John Wigley.
James Parsons Williamson.

JOHN THOMSON WILLIAMSON.
JOHN MICHAEL WILLIMAN.
ROBERT WILSON.
VALENTINE EDWARD THOMAS YORATH.

The Scrutineers reported that the following had been duly elected as

Associate Members.

FRANK FREEMAN BAYNE.
LIONEL JOHN BENTLEY.
LAURENCE PARKER BRUNT, B.Sc. (Eng.) (*Lond.*).
GEOFFREY HARVEY CALDICOTT, B.Sc. (*Edin.*).
ARCHIBALD PARKES CAMPBELL, B.E. (*New Zealand*).
JOHN VICTOR TELFER CAMPBELL, B.Sc. (Eng.) (*Lond.*).
BERNARD FRANCIS LEANDER DAVIS, B.Sc. (*Cape Town*).
DAHYABHAI SHIVA BHAI DESAI, B.Eng. (*Sheffield*).
STEPHEN VERRALL GARDNER.
ROBERT GILMOUR.
GLYN GOULD, Stud. Inst. C.E.
FRANK HARRISON HEATON, Stud. Inst. C.E.
LEWIS SAMUEL HIGHFIELD.
JOHN IREDALE HIND, Stud. Inst. C.E.
CLEMENT WALTER GODFREY HINDLEY, M.A. (*Cantab.*).
GERALD HERBERT HOLLOWAY, B.Sc. (*Edin.*), Stud. Inst. C.E.
WILLIAM MACKAY SINCLAIR HOUSTON, B.Sc. (*Glas.*), Stud. Inst. C.E.
ARCHIBALD CHARLES IVES, B.E. (*New Zealand*).
GOVIND DATTATRAYA JOGLEKAR, B.E. (*Bombay*).
GEORGE DAVIS KING, Stud. Inst. C.E.
WALTER GRAHAM BEATTIE LANGMUIR.
ROBERT LESLIE, B.Sc. (*Cape Town*), Stud. Inst. C.E.
WILLIAM HUBBLE LINDSAY, B.Sc. (Eng.) (*Lond.*).
DONALD JESSE MCCULLOCH.
WALTER LAWRANCE NOWELL MADELEY, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
ALEC MAPLETOFT, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
IAN ALASTAIR DUNCAN MILLAR, B.A. (*Cantab.*), Stud. Inst. C.E.
CECIL JAMES RAE, M.Eng. (*Liverpool*).
JOHN STANLEY RAINE, B.Sc. (*Durham*).
RICHARD REATCHLOUS, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
ROBERT HALL RINGROSE.
KENNETH ALBERT ROSE, Stud. Inst. C.E.
JOHN EDGAR SANDERS.
ALBERT THANGA RAJAH SARAVANAMUTTOO, B.Sc. (*Glas.*).
HAROLD FREDERICK SCRIMGEOUR.
RONALD STAMMERS.
WILLIAM ALEXANDER STOCKAN, B.Sc. (*Edin.*).
NEBHRAJ METHARAM THADANI, B.Sc. (*Edin.*).
ALEXANDER DUDLEY REID WATSON, B.Sc. (Eng.) (*Lond.*).
ROBIN THOMSON WICKHAM, B.E. (*W. Australia*).
GORDON NATHANIEL WILKINSON, Stud. Inst. C.E.

PRESENTATION OF THE JAMES ALFRED EWING MEDAL.

The President stated that the first business of the Meeting was one which gave him, The Institution, and the Council in particular, great pleasure, and that was to award the James Alfred Ewing Medal for 1940 to Mr. H. R. Ricardo, B.A., F.R.S., Assoc. M. Inst. C.E., who was well-known for his research work into the design of high-speed combustion engines and for his technical publications on that subject.

The Medal, the President mentioned, was founded in 1936 in memory of the late Sir Alfred Ewing, Honorary Member, and was awarded for specially meritorious contributions to the Science of Engineering in the field of Research.

Mr H. R. Ricardo expressed his appreciation of the award to him of this Medal, of which he was tremendously proud, and thanked the President for his very favourable and encouraging remarks.

JAMES FORREST LECTURE, 1941.

The President said that the James Forrest Lecture had been established and endowed in honour of Mr. James Forrest, who had been Secretary of The Institution from 1859 to 1896, and Honorary Secretary from 1896 until his death in 1917.

Mr. Forrest bequeathed to The Institution some pieces of silver plate which had been presented to him during the course of his life. That plate was normally exhibited on each occasion of the James Forrest Lecture, but it would be appreciated that, owing to war conditions, it was undesirable that the plate should be displayed that evening.

He had great pleasure in introducing the Lecturer, Professor E. N. da. C. Andrade, Ph.D., D.Sc., F.R.S., who had been Quain Professor of Physics in the University of London since 1928, Scientific Adviser to the Director of Scientific Research, Ministry of Supply, since September, 1939, and a member of the Ministry's Advisory Council on Scientific Research and Technical Development since its inception. He had been also for several years Professor of Physics in the Royal Artillery College, Woolwich. His numerous contributions to physical literature had received wide recognition, and he was Editor for Physics of the *Encyclopaedia Britannica.*

"The Mechanical Properties of Solids."

By Professor Edward Neville da Costa Andrade, Ph.D., D.Sc., F.R.S.

Introduction.

It is a strange paradox of science that the more gross and palpable the phenomena, the harder it is to win an understanding as to their true nature; that the ultimate mechanism of happenings which involve objects that we can see and handle readily is much more obscure and complicated than that of things concerning which our knowledge is only indirect. Scientifically the behaviour of gases is much more readily explained than that of liquids or solids, although man as an artificer has been dealing with the latter for hundreds, nay thousands, of years, and had amassed isolated rules as to individual behaviour before he knew of the existence of any gas but air. With gases themselves we find that the more tenuous they are, and in that sense the more difficult to handle, the simpler they are. The flow of gases at low pressure is simple, that of liquids more difficult, that of solids still more complicated. The flow of electricity through metals, the laws of which were worked out long before the flow of electricity through gases was studied seriously, offers much greater difficulties to the theoretical physicist than does the flow of electricity through rarefied gases.

The reason for this contrast is, of course, simple. It is easier to explain the behaviour of atoms and molecules, in terms of which our modern theories are constructed, when they are widely separated, so that for most of the time their action upon one another is negligible, than when they are packed close together, so that everything depends upon the forces which they exert upon one another.

The physicist who is interested in the theoretical and the predictable behaviour of electricity and the modern electrical engineer meet upon the common ground of electronic studies, which have to-day such important technical applications. The civil engineer and the physicist who is interested in the fundamental behaviour of matter, the atomic explanation of its mechanical properties, have so far scarcely come into touch, because precisely those properties of solids which confront the builder of bridges and the designer of dams—and which have, in some cases, confronted him for centuries—are those which the theorist finds the most intractable. Any craftsman can show the most experienced physicist things about the behaviour of iron, for instance, which it will puzzle him to explain. A piece of copper wire bent round the finger forms a smooth curve, whilst a piece of soft iron wire of the same dimensions makes sharp bends and takes polygonal form. Why? Orowan [1], who called attention to the phenomenon, has put forward a tentative explanation, it is true, but it only throws the matter back on phenomena that are still only partially understood.

THE MECHANICAL BEHAVIOUR OF SOLIDS.

What I want to discuss to-day is the mechanical behaviour of solids. I have already pointed out that the difficulty of furnishing a molecular explanation for this behaviour lies in the fact that, with solids, the proximity of the molecules makes the intermolecular forces of prime importance. In liquids the molecules are packed together about as densely as they are in solids, but there is one great and familiar difference between solids and liquids that is fundamental for our subject. With a liquid of given chemical composition the physical properties are perfectly definite and can be measured on different specimens with a consistency limited only by the precision of the measurements; with a solid of given chemical nature, say a pure metal, many of the properties may vary widely from specimen to specimen, according to the previous history of that specimen. In particular, the mechanical strength is not accurately specified when we have specified the substance and its degree of purity, and, as a further case of solid idiosyncrasy, certain rather complicated properties of ionic crystals depend markedly on the state of the specimen. Even the density depends somewhat on the treatment; thus while, in the Tables, the density

[1] Proc. Phys. Soc., 52, 8, 1940.

of mercury is given to within 1 part in 100,000, the values for the density of copper vary over a range of 4 per cent.

Some properties, however, of solids are quite consistent from specimen to specimen: we may cite specific heat, melting-point, coefficient of thermal expansion and lattice constants. Smekal [1] has called the first class of properties, those, like mechanical strength in particular, which vary widely from specimen to specimen, according to the history of the sample, "structure-sensitive", as against the "structure-insensitive" properties just cited. The existence of structure-sensitive properties in crystals shows one thing clearly, that actual crystals cannot consist of atoms or molecules arranged in the perfect pattern contemplated by the mathematician—on a perfect lattice, to speak in more technical language—nor can they be structures in thermodynamic equilibrium. If the crystals were perfect they would all have the same properties if of the same material; similarly, if they were in thermodynamic equilibrium, they would eventually, from whatever arrangement they started, reach a final arrangement corresponding to the least free energy, and this arrangement, and consequently the mechanical properties, would always be the same. All crystals as usually dealt with must, then, be imperfect, although the possibility of preparing a perfect crystal must not be definitely excluded. The essential difference for our purposes is that a solid can support internal stresses, which may be different from specimen to specimen, while a liquid, by definition, cannot support any internal stresses permanently.

The Behaviour of Crystals.

Let us consider how a perfect crystal should behave under stress. Each atom is in equilibrium because both attractive and repulsive forces exist between any two atoms, varying inversely as different powers of the distance between them—say an attractive force varying as $1/r^2$ and a repulsive force varying as $1/r^n$, where n may be somewhere about 10. At the equilibrium distance these forces just counterbalance; if we try to separate the atoms the attractive force preponderates, while if we try to push them together the repulsive force preponderates. If we try to slide one plane of atoms over another plane, or, say, one row of atoms over another, the distance between two adjacent atoms first increases and then decreases again; the force will rise to a maximum and then decrease. If, therefore, we apply an increasing tension normal to a given set of crystal planes, the distances between the atoms should increase until the tension is sufficient to overcome the attractive forces, when the planes should separate and the crystal should break. Until this point is reached the crystal should, if we release the tension, always return to its original state. We should expect a range of perfect elasticity, followed by sudden fracture.

[1] *Phys. Z.*, 34, 633, 1933.

For a shearing stress we should likewise have a region of perfect elasticity, but if a stress be sufficient to cause the plane to move one-half of one atomic step with reference to the neighbouring plane, then it should be sufficient to cause it to execute any number of steps. A shearing stress exceeding a certain value would produce indefinite glide of one plane over the other, without, of course, any hardening.

Actually, as we shall see, nothing of this kind occurs. Before passing on, however, we will inquire how a rough theoretical estimate of mechanical strength can be made. When we break an ideal crystal rod in tension two fresh surfaces are formed, and if the broken pieces of crystal are identical in structure with the original crystal the only increase in energy will be represented by the energy of these surfaces. This must be derived from, and cannot exceed, the elastic energy stored in the crystal at the moment of breaking: if F is the breaking stress, d the extension at breaking-point of a length equal to the intermolecular distance a, γ the surface tension, and if we consider unit cross-sectional area, then, if Hooke's law holds up to fracture,

$$\text{work done} = \tfrac{1}{2}Fd \gtrdot 2\gamma\,;$$

and if E is Young's modulus,

$$F = \frac{d}{a}E,$$

so that

$$F^2 \gtrdot \frac{4\gamma E}{a}$$

To find F we have to estimate the surface tension of a solid. This cannot be measured by ordinary methods, but, since the intermolecular distances concerned in the solid state are not very different from those in the liquid state, the intermolecular forces must be much the same in both cases and the surface tension of a substance in the solid state cannot be very different from that of the same substance in the liquid state. For liquid metals the surface tensions are of the order of 500 dynes per centimetre[1], the intermolecular distance is about 3×10^{-8} centimetre and Young's modulus, for the stronger metals, about 2 megabars[2]. This

[1] The recorded figures vary from 350 dynes per centimetre for antimony at 640° C. to 1,150 dynes per centimetre for a certain cast iron at 1,150° C. For mercury at 20° C. the figure is 465 dynes per centimetre.

[2] Strength is expressed in various units, such as tons weight per square inch; pounds weight per square inch; grams weight per square centimetre; grams weight per square millimetre; kilograms weight per square millimetre ("weight" being habitually, but unfortunately, omitted); dynes per square millimetre, and so on. The best unit for scientific work would seem to be the *bar*, which is 10^6 dynes per square centimetre: it is already habitually used by the meteorologists to express pressure. 10 grams weight per square millimetre is (within 2 per cent.) 1 bar; 1 ton weight per square inch is 154·6 bars. The kilobar is, of course, a thousand bars: the megabar a million bars. I hope that engineers will consider the gradual adoption of the system. 1 kilobar is about 6·5 tons weight per square inch; 1 bar is about 14·5 lb. weight per square inch.

gives for F a value of about 360 kilobars (= 2,300 tons weight per square inch), which is about forty times the tensile strength of the toughest metal[1]. If we take the yield-point, instead of the tensile strength for comparison with the theoretical value, the discrepancy is even greater. Similar calculations can be carried out for the theoretical shear strength, and give a value of about 130 kilobars, some forty times that observed for steel. Thus even the toughest metals show much less than the theoretical strength.

The ordinary metals, however, are polycrystalline metals; it may be suggested that for comparison with theory it would be fairer, perhaps, to consider a single crystal. I take rock salt as an example, for it is a simple face-centred cubic crystal composed of alternate positive and negative ions, for which the mathematicians can carry out exact calculations, of, for example, the theoretical strength. For rock salt this works out to be 20 kilobars and rupture should theoretically be preceded by an elastic extension of 14 per cent. It is interesting to note that surface-tension considerations give a similar value of the theoretical strength, which confirms our belief in the validity of the surface-tension method of estimating strength. The actual breaking strength of rock salt under tension is, however, about 20 bars, that is, about one-thousandth of what it should be. Rock salt does give brittle fracture, with very small extension, at ordinary temperatures, although at higher temperatures, above 600° C., it shows plastic behaviour.

A perfect ionic crystal of a type which is well understood theoretically is, then, in practice much too weak and has much too small an elastic region. Let us look at its behaviour in a little more detail, for it will help us later with our metals. To examine a transparent crystal we have an agent which we cannot use for metals, polarized light. A cubic crystal such as rock salt has, in an unstrained condition, no effect on such light; any strain that makes the crystal depart from its precise cubic structure causes it, however, to become doubly refracting. Obreimow and Schubnikow[2] loaded a crystal of rock salt in a beam of polarized light; at a low stress, of about 8 bars, bright streaks appeared along two directions at right angles, which were the traces of (110) planes. These lines did not appear as soon as the stress was applied, but arose spontaneously some 20 seconds later. As the stress was increased the bright lines increased

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[1] I prefer not to use the term "ultimate tensile strength", since it is usually employed in a very unfortunate way by engineers. They ordinarily denote by it the breaking load divided by the original cross-sectional area, whereas, except in the unusual case of brittle fracture, marked reduction of area ("necking") precedes rupture. The term, used in this conventional manner, has no significance, and its use has led to much confusion. In the case of plastic flow of metals I showed some time ago the great simplification that results from measuring the flow under constant stress instead of under constant load.

[2] *Z. Physik*, 41, 907, 1927.

in number, but appeared always in the same crystallographic directions. Surface marks also appear in the same direction if the crystal undergoes plastic extension at high temperature. The crystal shows permanent slip on certain crystallographic planes, but the polarized light shows that its crystalline perfection has been destroyed along these planes. The glide-planes were established much earlier in metals, but I quote rock salt first because of the ease with which this latter point can be established. The same thing has been shown with other transparent crystals, for example, potassium halides.

Glass: Surface Effects.

Let us see if we fare any better as regards strength if we take a non-crystalline substance, say glass. The surface tensions of molten glasses are round about 150 dynes per centimetre, which gives theoretical strengths of the order 100 kilobars. The actual strength of ordinary glass fibres, which give brittle fracture at room temperatures, is from 300 to 900 bars —less than one-hundredth of what it should be. Here, however, we meet a curious fact of great importance for our subject: freshly drawn glass fibres, of hard glass, are very much stronger than old fibres, of the same glass, but gradually lose their strength as time goes on and come down to a steady value. Touching or handling the fresh fibres weakens them. A. A. Griffith [1], who has carried out fundamental work on this subject, assumes that the cause of the weakening is the formation of invisible surface cracks, which are absent in a fresh fire-drawn material. The breaking strength will depend upon the depth of the crack and the radius of curvature at the end: the greater the former and the smaller the latter, the higher the local stress and the weaker the material. There is a limit to the radius of curvature: it cannot be less than something of the order of the inter-atomic distance. If we assume a reasonable value we can work out the depth of the crack needed to give the observed strength, and it comes out about 1 or 2μ, say twice the wavelength of visible light. Of course if we have threads of this order of diameter we cannot have cracks of this depth, and very fine threads are, in fact, stronger per unit cross-section than large threads. The variations of stress, due to cooling, necessary for the formation of the cracks probably cannot be established in such fine threads.

These cracks are no longer a mere hypothesis. Mr. Tsien and I [2] found that attack by sodium vapour (but not by hydrofluoric acid) would develop the cracks and make them visible. On freshly drawn hard glass tube no, or very few, cracks could be detected—on the same tube, when old, many cracks were made visible. They ran transverse to the direction of drawing of the glass rod or tube, as they should. Various controls established that

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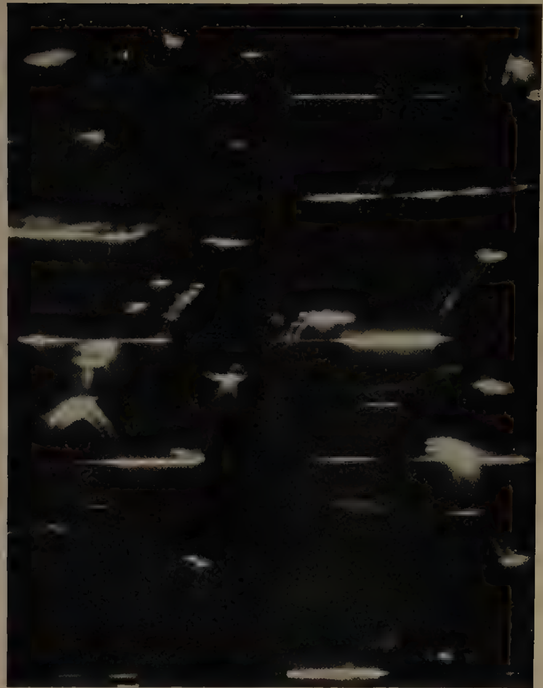
[1] Phil. Trans. Roy. Soc., A 221, 163, 1921.
[2] Proc. Roy. Soc., A 159, 346, 1937.

Fig. 1.

× 80

TYPICAL SURFACE CRACKS IN PYREX GLASS, DEVELOPED BY SODIUM VAPOUR.

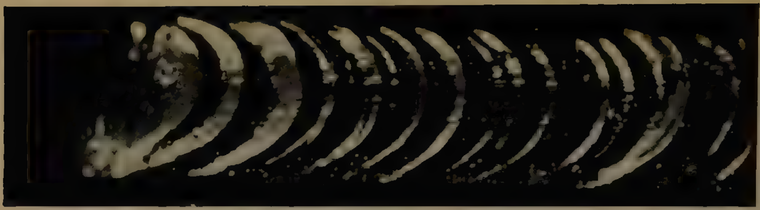
Fig. 2.



× 130

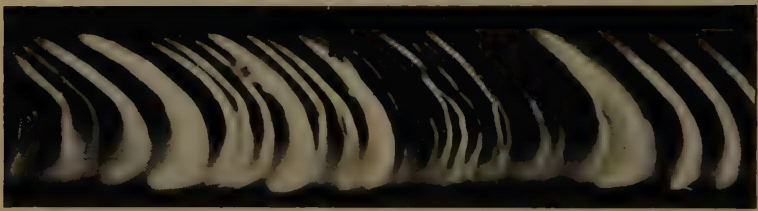
TYPICAL CRACKS IN QUARTZ GLASS, DEVELOPED BY SODIUM VAPOUR.

Fig. 5.



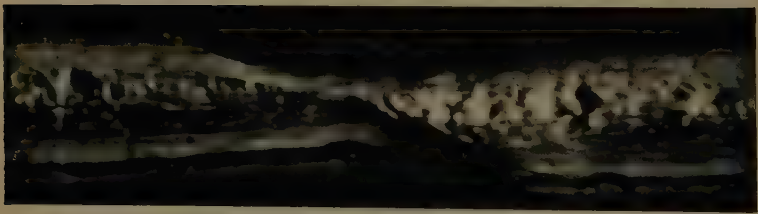
SLIP PLANES OF MOLYBDENUM AT 1000° C.

Fig. 6.



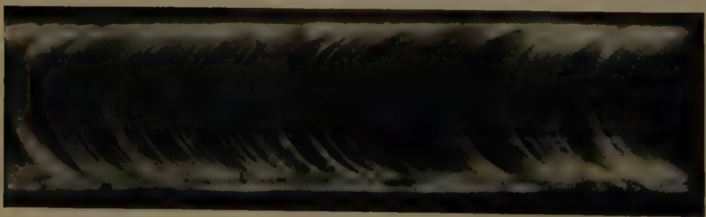
SLIP PLANES OF MOLYBDENUM AT 1500° C.

Fig. 7.



SLIP PLANE OF MOLYBDENUM AT 2000° C.

Fig. 8.



SLIP PLANES OF CADMIUM AT ROOM TEMPERATURE.

we were not dealing with a spurious phenomenon. *Fig. 1* shows typical cracks, developed by sodium vapour, on the inside of a pyrex glass tube: *Fig. 2* shows the same kind of thing with quartz glass. Dr. Martindale and I [1] also found that when very thin films of gold and silver were heated to about 300° C. on glass surfaces minute crystallites formed up in lines, which were in general parallel. These lines are not caused by visible scratches but are attributable to minute surface cracks, a hypothesis which is confirmed by the behaviour of the two known types of diamond, one of which is perfect while the other contains invisible flaws, made evident by certain physical properties. The perfect type shows no lines of crystallites; with the other type such lines form. I may add that just before the war I succeeded in showing the minute surface cracks in hard glass by a special type of illumination [2].

An experiment of Orowan's [3] is instructive in this connection. If a strip of mica be loaded by means of parallel metal clamps, extended across the whole strip, so that the edges are stressed, we find a maximum strength of about 3 kilobars, approximately that of wrought iron and very much in excess of that of glass and of rock salt. The flat faces of mica are very perfect [4], but the cut edges have, of course, no such perfection. By using special clamps, gripping the sheet near the middle only, stress at these edges can be avoided. For a mica sheet loaded in this way the strength goes up to 32 kilobars, more than ten times that with the other method of loading, and much in excess of that of the best constructional steel. The prejudicial effect of trivial and invisible cracks at the cut edge is clear.

These considerations furnish us with fundamental material for the understanding of brittle rupture, but possibly appear very remote from any practical consideration. They may, however, not always be so. Recently a serious proposal to use ordinary glass as a substitute for steel in reinforced concrete has been put forward by Messrs. Soden and Lincoln [5]. The glass is introduced in the form of flat "planks" set on edge in the beam, as shown in *Fig. 3*, the lower edges being in tension and the upper edges unstressed or in slight compression [6]. Now the inventors specify that the lower edge must be a fire-finished edge, while the upper edge, which is cut, must not be in tension. The minute cracks in a cut edge are certainly much more pronounced than those in a fire-finished edge. Glass-reinforced beams have shown themselves perfectly satisfactory under static load, but offer some disadvantages under impact test, as compared

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1 Phil. Trans. Roy. Soc., A 235, 69, 1935.

2 Unpublished.

3 *Z. Physik*, 82, 235, 1933.

4 They show no lines of crystallites when metal films are heated on them.

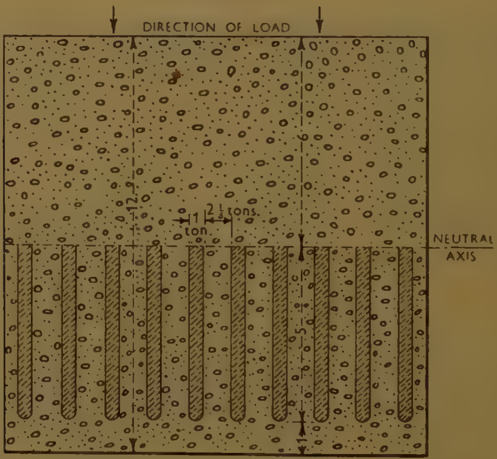
5 Engineering, 150, 225, 1940.

6 The inventors take for the safe stress in glass 170 bars (2,500 lb. per square centimetre), about one-half of the worst figure quoted for tensile strength and one-fifth of the best—the tensile strength of the particular glass used is not given.

with ordinary reinforced concretes. You will not expect me to pronounce definitely as to the feasibility of utilizing this new invention, but the proposal is evidently worthy of serious consideration.

There are other industrial examples of the importance of the avoidance of surface cracks. Toughened glass is prepared by rapidly cooling the surface of the slabs while the interior is still molten. When the internal part solidifies and cools the shrinkage pulls the surface into a state of compression, which prevents the formation of cracks. The glazing of industrial porcelain, for insulators and such like, is also instructive in this connexion.

Fig. 3.



Concrete Reinforced By Glass.

If the object is to be strong the glaze must have a coefficient of thermal expansion lower than that of the porcelain body. When the cooling takes place after glazing, the greater shrinkage of the body pulls the surface glaze into a state of compression. If a glaze with a higher coefficient of expansion be used, the strength of the porcelain is less than that of the unglazed body. The swelling of the metal surface by the process of nitriding may also act as a talisman against fatigue cracks, which usually start at the surface.

The study of ionic crystals, then, has shown us that yield takes place along certain crystallographic planes, and that along these planes a state of permanent internal stress is set up. The study of glasses and of mica has shown us that submicroscopic imperfections, invisible cracks, play a fundamental part in the yield with brittle fracture. I may add that a certain peculiar behaviour of rock salt has been invoked to support the theory that surface cracks are the origin of glide. If a plate of rock salt be held under warm water it can be bent like a piece of lead, and in general

shows a plastic behaviour which contrasts strongly with its brittle behaviour under ordinary conditions. Joffe [1], who directed attention to the effect, attributes it to the solution of the surface layers and, in consequence, the removal of surface cracks; others have explained it as due to the penetration of water into the crystal structure. I have shown [2], however, that a crystal can be readily bent under cold running water; this, owing to the avoidance of any approach to saturation, will give a high rate of surface solution, but a smaller diffusion than warm water, and so speaks for a surface effect.

The Mechanical Behaviour of Metals.

I now turn to the question of the mechanical behaviour of metals. Metals, as the engineer knows them, consist of masses of crystals, separated by boundaries that appear sharp under the highest magnification, and the study of the crystal structure of metals, initiated 77 years ago by Sorby, is well known to be of the greatest importance for the metallurgist. As we have seen, the behaviour of a perfect single crystal under stress should theoretically be either brittle fracture, within the elastic limit, under tension, or unlimited glide under shear, and the stresses theoretically required to produce fracture or glide are very much in excess of those observed experimentally. Further, an ideal crystal should show no hardening under plastic flow.

At first it might seem possible that the failure of a piece of ordinary polycrystalline metal to show the properties of a perfect crystal could be attributed to the fact that it is not a single crystal, but an aggregate. The crystal boundaries might play an important part in determining the metallic properties. We shall see that they do, but hardly in the way to be anticipated. In any case, it is clear that if we desire to understand metallic behaviour we must first understand the behaviour of a single crystal of metal, just as any civil engineer who wanted to understand the properties of a girder structure would first study the properties of a single girder.

Single crystal specimens of metals can be prepared in a variety of ways. If one desires to experiment with a rod of crystal such as rock salt, it is usual to take a block of crystal, with the ordinary faces developed, so that it is clearly recognizable as a crystal, and to turn or cut a specimen from it. This is not the usual way of preparing single crystal specimens of metal, although K. W. Hausser [3] did prepare crystals of metal weighing several pounds, and turn, for instance, spheres out of them; even then the original specimens were made in a cylindrical mould and did not have natural faces. It is difficult to prepare such large specimens, and the

[1] *Z. Physik*, 22, 286, 1924.
[2] Unpublished.
[3] *Wiss. Veröff. Siemens-Konzern*, 5, 144, 1927.

cutting of the test-piece is likely to introduce strains which, as we shall see, modify the properties. The better and more usual way is to prepare the specimen from ordinary polycrystalline metal, and then, by suitable treatment, to change its inner structure to that of one crystal, leaving the outer faces unmodified. Sir Harold Carpenter and Miss Elam [1], for instance, cut flat test-pieces of aluminium, of the ordinary form, and then by a process of repeated strain and reheating converted the whole piece into one crystal, or occasionally into two or three crystals with boundaries. The more usual form, however, is that of a cylindrical rod or stout wire.

I prepared the first single crystal specimens of certain metals by the suitable heating of wires as long ago as 1913; but the work was interrupted by the War of 1914 and it was not until nearly 20 years later that I was able to take up the subject again. A usual method is to cause the desired crystallization by melting the wire and then, by suitable progressive cooling from one end, promote the growth of the crystal structure from a single small "seed" crystal. There are many modifications of the method, with which I must not trouble you now.

The specimen so prepared will, to casual inspection, be indistinguishable from a wire or rod of ordinary metal. X-rays, of course, will at once reveal its nature, as all the reflecting planes, of one kind, will be parallel (or very nearly parallel) to one another. There is no need, however, to use X-rays, valuable as they are as a method of investigation, to show that a single crystal specimen is not an ordinary wire; the simplest mechanical tests will at once give evidence that there is something peculiar about it.

In the first place metal single crystals are not, as might be expected, very strong but they are very soft. They also show very remarkable strain hardening, a phenomenon which I have not hitherto discussed. A copper crystal rod half an inch in diameter behaves like lead at first manipulation; it can be very easily bent in the hands, but becomes progressively harder to deform. It takes a very strong man to bend back to its original form such a rod that has once been deformed into a semicircle. Brittle behaviour is not unknown in single crystal specimens (bismuth crystals, for instance, show brittle fracture at low temperature), but it is not typical.

The Geometry of Glide.

Remarkable properties of single crystal wires can be exhibited by simply subjecting them to tension. On slight extension a number of parallel markings appear on the surface and as elongation proceeds these become, in general, more and more marked: in some cases the number increases. The wire retains practically its full diameter in one direction

[1] Proc. Roy. Soc. A, 100, 329, 1921.

and thins in a direction more or less normal to this, so that in the case of wires that can be pulled to several times their original length, such as, for example, cadmium wires, the original cylinder becomes a thin ribbon. As the result of the work of Polanyi, Schmidt, G. I. Taylor, and others this behaviour has been expressed in terms of the geometry of the crystal [1]. The slip takes place preferentially on certain types of crystal planes and in certain definite crystallographic directions—the glide planes and glide directions.

Conditions are simplest in the metals of hexagonal crystal structure, for here the usual glide plane [2] is the hexagonal base, which is unique. In face-centred cubic crystals, on the other hand, glide takes place on the octahedral faces, of which there are four, all exactly equivalent to one another. Considering, then, for the moment hexagonal metals, such as zinc, cadmium and magnesium, the slip takes place on one set of planes, all parallel to one another. As regards direction, it takes place along a line joining opposite corners of the hexagon, so that there are three equivalent glide directions. Which of these three is effective depends upon the direction in which the force is applied. *Fig. 4* [3] illustrates this schematically: (*a*) shows the wire before slip, the plane exposed being structurally a hexagonal basal plane, with the directions of the three digonal axes indicated by the hexagon. The digonal axis nearest to the direction *T* of the tension, projected on the plane, will be the one along which glide takes place; (*c*) shows the situation after glide; (*b*) and (*d*) are views of (*a*) and (*c*) respectively, seen from a line in the glide plane and normal to the wire axis.

If slip took place equally readily on all parallel atomic planes the crystal surface would, of course, be as smooth after extension as it is before extension. What takes place, however, is a preferential glide in the neighbourhood of certain planes, the so-called slip planes, spaced at more or less regular intervals, so that the crystal appears to slip in parallel slices or slabs. This produces the remarkable stepped appearance characteristic of crystal wires that have been much extended. In general the spacing of the slip planes becomes wider and wider as the temperature is raised. With molybdenum at 1,000° C. and 1,500° C., for instance, the coarse appearance of the slip packets is very striking, as seen in *Figs. 5* [4] and *6* [5]; with molybdenum at 2,000° C. the slip may take place all at one plane as shown in *Fig. 7* [5]. This may be compared with the appearance of cadmium at room temperature, *Fig. 8* [6]. It has been suggested that the

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[1] See, for example, C. F. Elam, "Distortion of Metal Crystals," Oxford, 1935.

[2] Under certain conditions of temperature glide can take place on other planes, the so-called prismatic faces of the first and second kind.

[3] Adapted from E. Schmid and W. Boas, *Kristallplastizität*, A 59, Berlin, 1935.

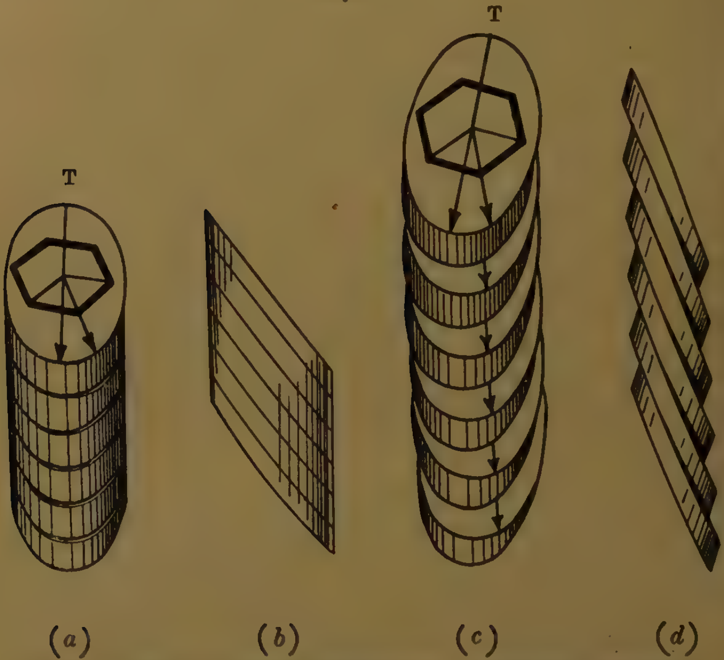
[4] L. C. Tsien and Y. S. Chow, Proc. Roy. Soc. A, 163, 19, 1937.

[5] E. N. da C. Andrade and Y. S. Chow, Proc. Roy. Soc. A, 175, 290, 1940.

[6] L. C. Tsien and Y. S. Chow, *loc. cit.*

choice of the particular region where marked glide takes place may be due to local impurities, but experiments carried out in my laboratory with wires of exceedingly pure solid mercury [1] (the impurity was probably about 1 in a hundred million), show that this cannot be so. The slip bands are very fine and close. *Figs.* 9 [2] and *10* [3] show photographs, due to Dr. Greenland, of slip in wires of solid mercury, the former viewed from a direction in the glide plane and normal to the wire axis. The whole question of the

Fig. 4.



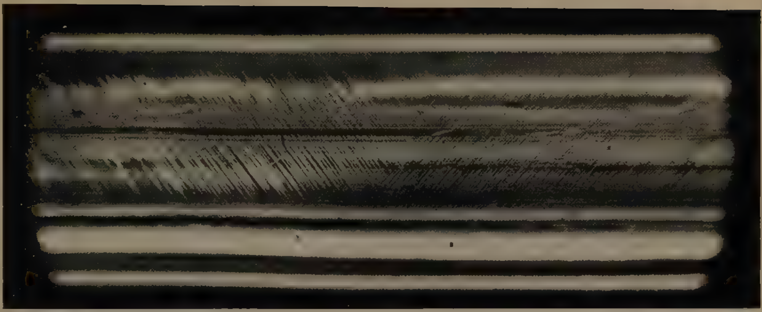
Glide Plane and Glide Directions in a Hexagonal Crystal.

formation of these bands, which represent a kind of local avalanches, is a complicated one to which reference will be made later.

If a number of single crystal wires of cadmium, say, are taken, it will be found that the load under which they begin to deform will vary from specimen to specimen. The reason is that what determines the plastic behaviour is the shear stress on the glide plane resolved in the glide direction ; thus, if a load W be applied to a wire specimen of cross-sectional area A, and the glide plane and glide direction make angles χ and λ with

[1] Mercury can be prepared in greater purity than any other metal.
[2] E. N. da C. Andrade and R. Roscoe, Proc. Phys. Soc., 49, 152, 1937.
[3] K. M. Greenland, Proc. Roy. Soc. A, 163, 28, 1937.

Fig. 9.

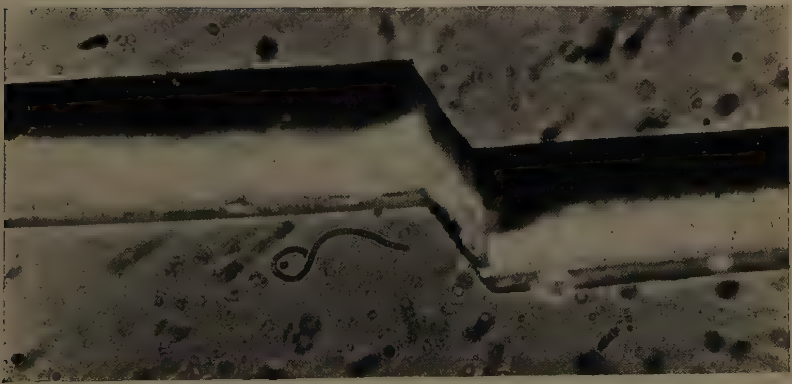


SLIP BANDS OF PURE MERCURY.

Fig. 10.

SLIP BANDS OF PURE MERCURY.

Fig. 11.

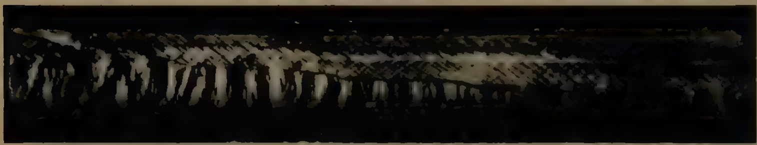


SLIP IN CADMIUM CRYSTAL WITH PLANES NEARLY NORMAL TO AXIS.

Fig. 12.

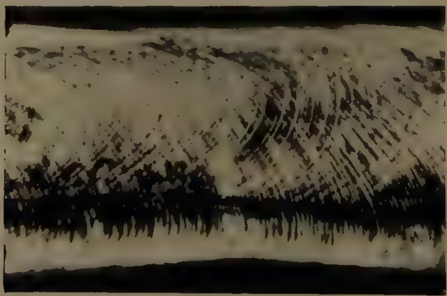
DOUBLE GLIDE IN CADMIUM: TWO DIRECTIONS EQUALLY FAVOURABLE.

Fig. 13.



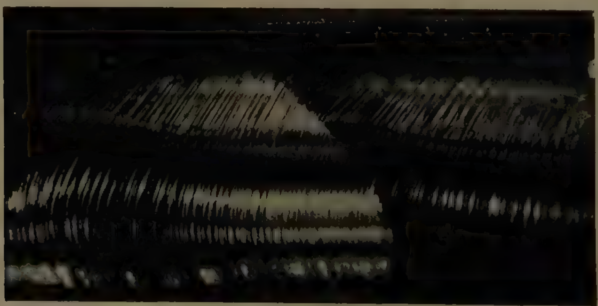
DOUBLE GLIDE IN MOLYBDENUM AT 300° C.: TWO CRYSTALS SIDE BY SIDE.

Fig. 14.



DOUBLE GLIDE IN MERCURY: TWO PLANES EQUALLY FAVOURABLE.

Fig. 15.



TWINNING IN A MERCURY CRYSTAL.

the axis of the wire, then the so-called critical shear stress, which is a constant for the metal, is given by

$$S = W/A \cos \lambda \sin \chi$$

The behaviour is complicated by the fact that as the wire is stretched the planes rotate: they tend to move all parallel to one another, and can only do this by bending near the grips. The geometry of glide has been fully worked out, and explains some of the extraordinary features which single crystals show under stress. For instance, if the glide planes of cadmium are nearly normal to the axis of the wire, glide may take place at one or two places only, giving the strange appearance shown in *Fig. 11*[1], which I call "stove-pipe slip." The main part of the wire bends very slightly so as to keep the mid parts of the short elbows in line with the direction of tension. Another strange appearance is shown in *Fig. 12*[2]; this takes place when two of the glide directions are equally favourable, so that short lengths of the crystal slip alternately in one and the other direction. Double glide in cubic crystals and other crystals where there is more than one set of equivalent glide planes can also take place on different sets of glide planes, which either happen to be equally favourably disposed initially, or become so in consequence of changes in the course of extension. *Fig. 13*[3] shows a curious case, where two crystals have grown side by side in a molybdenum wire, and on extension at 300° C. double glide has taken place in each. *Fig. 14*[4] shows double glide in a mercury crystal. Double glide on two sets of planes, disposed equally favourably, can be also seen in *Fig. 9*.

After a certain amount of extension has taken place the crystal may show another phenomenon, that of twinning, the molecules moving so that the crystal structure in one part of the twin is the mirror image of that in the other part of the twin. *Fig. 15*[5] shows a boundary where twinning has taken place in a mercury crystal. The reason for this twinning is still obscure, but it adds to the difficulties of the subject. In cubic and rhombohedral metals, where there are groups of exactly equivalent glide planes, on which glide may take place simultaneously or consecutively, things may be very complicated. In a general way, however, the geometry of the glide in single crystals has been worked out and most of the broad phenomena can be explained. If it is remembered that the individual crystallites in a polycrystalline metal can behave in any of the ways indicated, and that crystallites differently disposed behave in different ways in one specimen, the complexity of the problems of an ordinary metal can easily be realized.

---

[1] K. M. Greenland, *loc. cit.*
[2] E. N. da C. Andrade and R. Roscoe, *loc. cit.*
[3] E. N. da C. Andrade and R. Roscoe, *loc. cit.*
[4] E. N. da C. Andrade, unpublished.
[5] E. N. da C. Andrade and P. J. Hutchings, Proc. Roy. Soc. A, 148, 120, 1935.

Strength of Metal Single Crystals.

When we turn to the question of strength we enter a very difficult field. The simplest case is that of brittle fracture. The determining factor here is the tension normal to the plane of rupture. If the critical tension N which leads to fracture; the critical shear stress S; and the crystal structure are such that a crystal can be so disposed that the stress resolved normal to the plane of rupture is N while the stress resolved along the glide direction is less than S, fracture without glide can take place. With crystals of hexagonal structure it is possible to realize these conditions, and we find the recorded critical tensions run from 120 to 18 bars with different metals and different temperatures, for instance, 20 bars for the basal plane of zinc at $-185°$ C. This compares with 1·3 kilobar for polycrystalline zinc and a theoretical value of some 100 kilobars.

Brittle fracture is, however, rather the exception than the rule. For metals of cubic structure, where there are many alternative equivalent glide planes, the values of N and S and the geometry of the crystal apparently prohibit it. Even with hexagonal metals glide is the normal occurrence.

The inception of glide, which takes place by sudden avalanche-like slipping in the neighbourhood of isolated planes, occurs at very low stresses, of the order of 20 bars, much less than one-thousandth of the theoretical strength. If we wish to consider rupture, we are confronted with a variety of processes that can take place before actual fracture. We have in general a progressive hardening as movement proceeds, but quite often, especially with hexagonal metals, we ultimately get twinning, leading to glide on a fresh set of planes at a new inclination, which is followed by fracture. In the case of hexagonal metals, where brittle fracture can be brought about by suitable geometrical conditions, the glide which takes place before rupture can either raise or lower the breaking strength, according to temperature. For these metals the breaking strength is, however, of the same order as that in the case of brittle fracture. In the case of certain metals which will be mentioned later glide can lead to very great hardening, so that stresses of nearly a hundred times the critical shear stress can be supported, but here, as will be pointed out, the result of glide is that the specimen has lost much of its monocrystalline character. It remains true that a single crystal is very soft.

Before we look at any further facts, it may be well to consider the attempts that have been made to explain theoretically the outstanding difficulties offered by the softness of the crystals—their proneness to glide under very small shear stresses. The atoms are bound in their places by interatomic forces; the potential of the field of force having a minimum at the position where the atom is in equilibrium. If an atom is moved from this position the force tends to restore it; the case is similar to that of a table covered with a regular network of depressions, with a ball lying

in each depression. Now consider the force required to move all the balls at once in a given direction, so that each rises out of its hole and falls into the next hole in that direction. It will clearly be the force required to move a single ball multiplied by the number of balls, and this is equally true when a ball lies in every hole and when a few vacant holes are included in the array. The low value of the critical shear stress cannot be accounted for on the basis of the simultaneous movement of all atoms in a glide plane.

The position may perhaps be clarified by considering simply a row of cylinders touching one another, with a second row of cylinders resting on them, as in a model which I have constructed (*Fig. 16*). The force required to move one whole row over the other is considerable. Suppose, however, that the number of cylinders in the upper row is one or two fewer than that in the lower row. A place of non-fit, or dislocation, indicated by a white line above it, is arranged at one end of the row, towards the right, there being four in the upper row to five in the lower row here. To make the model resemble the atomic case all the upper cylinders in the model are connected by a long strip of elastic band. If we now run our hand over the upper row, from right to left, which is equivalent to imposing a force from left to right on the lower row, it is easy to make this dislocation travel along: at any given moment we are only moving a few cylinders through the periodic field of gravitational potential. After the dislocation has travelled, however, every cylinder has moved one place and the whole row is displaced relatively to that below, although at any given moment we were only moving a few cylinders.

The fact that glide takes place at shear stresses so far below the theoretical value shows that a whole sheet of atoms cannot travel at once over a neighbouring sheet. The conception of the glide mechanism, which we owe to Polányi, G. I. Taylor, and Orowan, postulates that in the metal crystal there are small localities in which the crystal is out of joint, places of misfit something like that represented very crudely in our model. Under shear the whole crystal plane does not move by one atomic spacing simultaneously: rather the misfit runs easily through the crystal, leaving in its wake atoms properly spaced. In a sense, then, the weakness of the crystal is due to imperfections which are somehow inherent in the structure. A perfect crystal might have theoretical strength, but probably no such thing exists.

There is considerable doubt and diversity of opinion as to the origin and end of the dislocations. G. I. Taylor [1], for instance, when proposing a very suggestive but rather formal two-dimensional model considers that the stress produces in the metal crystal a multitude of regularly spaced dislocations, which run a certain distance and then end their course in flaws which he postulates. It is difficult to see on this picture anything

[1] Proc. Roy. Soc. A, 145, 362, 1934.

to prevent the effect being reversible, which it certainly is not. Orowan considers that the dislocations are created at microscopic "Griffiths cracks", sharp notches where the stress has values considerably above the average, and that they run right across the specimen. W. G. Burgers and Kochendörfer believe that the crystal is built up of small blocks in which the orientation of the crystal structure differs slightly from block to block, for which there is X-ray evidence with rock salt and some other crystals. The dislocations are then supposed to start from the boundary of the blocks: how they are stopped is not clear. I have not time to discuss the details and difficulties: the general conception of a travelling misfit seems, however, to be undoubtedly the proper basis for any theory of glide.

Another fruitful conception is that of cracks, notches, or other regions of local peculiarity leading to increased local stress. Thinking of a wooden plant we may, perhaps, be allowed to call these hypothetical regions "knots." If these regions of stress concentration are small the stress will depend upon the irregularities of atomic motion in the neighbourhood —the so-called temperature fluctuations, which are the more violent the higher the temperature. Every now and then the atoms so will dispose themselves that, speaking loosely, the crack is sharper: the longer one waits the greater the chance that the local stress reaches some high critical value. This conception is due to Becker [1]: it has been elaborated by Orowan. The rate of flow of a crystal under stress will be proportional to the frequency with which the critical stress is reached at the local "knots." This leads to the following formula for the rate of glide, u :—

$$u = Ce^{-V(R-qS)^2/2GkT},$$

where R is the true critical shear stress, S the average externally-applied stress, q a factor to represent the increase at a "knot", G the shear modulus, k Boltzmann's constant, T the absolute temperature, and V a certain small volume, full consideration of which would take us too far.

This formula has certain interesting consequences. It shows the rate as depending very strongly on the stress: according to it, flow should take place at any stress, but if the stress is below a certain value the flow is unobservably small. This is the kind of thing we must expect to find in theories based upon probabilities.

If this view is correct, we cannot define a critical shear stress for a single crystal by saying it is the resolved shear stress at which glide begins to take place, for this will depend upon what rate we are prepared to consider noticeable. I have suggested that we define it as that at which a rate of 1 per cent. per hour takes place. This seems a convenient arbitrary figure, but it is nothing more. The exact rate does not make much difference: in the case of very pure cadmium, considered by Roscoe and myself [2],

[1] *Phys. Z.*, 26, 919, 1925.
[2] Proc. Phys. Soc., 49, 152, 1937.

Fig. 16.

MODEL ILLUSTRATING HOW GLIDE CAN TAKE PLACE BY THE PROPAGATION OF A MISFIT.

Fig. 17.

LAUE PHOTOGRAPH OF AN UNSTRAINED SODIUM CRYSTAL.

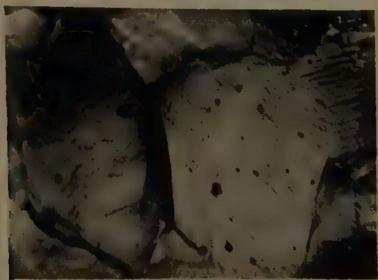
Fig. 18.

LAUE PHOTOGRAPH OF HEAVILY STRAINED CRYSTAL OF α-IRON.

Fig. 19.

ASTERISMS DUE TO STRAIN IN A SODIUM CRYSTAL.

Fig. 20.



SLIP IN STRAINED POLYCRYSTALLINE LEAD.

stresses of 1·5, 1·3, and 1·0 bar gave rates of 1 per cent. per 13 minutes, per hour, and per 7 hours respectively. A stress of 0·5 bar still gave a flow, but of only 1 per cent. in 13·5 days. In the neighbourhood of the suggested rate, with cadmium, for instance, the rate is doubled by about an 8-per cent. increase in the stress.

This way of regarding critical shear stress as the stress at which a determined slow rate of glide begins also explains a remarkable fact, as pointed out by Orowan, who has emphasized the importance of concentrating on the rate of flow as a property of the single crystal. While the rate of flow at a given stress increases very rapidly with the temperature, at a given stage of the flow, and the strain hardening is also very susceptible to temperature, the critical shear stress is not very much influenced by temperature: even at $-235°$ C. it is, for cadmium, only four times as big as it is a few degrees below the melting-point of the metal. If we take the critical shear stress to correspond to a fixed value of u at all temperatures this means that $(R - qS)^2/T$ is approximately constant, which gives $S = A - B\sqrt{T}$, a relation which agrees quite well with experiment.

The difficulty of interpreting curves showing the shear stress plotted against extension ("hardening" curves) will now be clear. If there is a time element, if flow is taking place all the time, these curves will depend upon the rate at which stress is increased. By working within a region where the rate of flow is very small, and applying the stress fairly rapidly, hardening curves with some meaning can be obtained, but sufficient attention has not always been given to this point. The whole question is a troublesome one, complicated by the effect of recovery, that is, a crystal that has been hardened by strain becomes soft again if left to itself at a sufficiently high temperature: for "soft" metals, like cadmium, room temperature is sufficient. Some workers have attributed the time flow entirely to recovery taking place while the stress is still applied: others have tended to neglect it altogether. Actually true flow in the absence of recovery undoubtedly exists, as can be demonstrated by working at low temperatures, where the rate of recovery must be so exceedingly slow as to be quite negligible: on the other hand recovery undoubtedly plays a part in flow at higher temperatures. Hardening at high temperatures and at low temperatures exhibits different features and requires rather more careful discrimination than has generally been made. The problems of strain hardening can, however, only be glanced at here.

The time element being eliminated as best one can, curves showing the variation of stress with glide[1] at temperatures suitably chosen can be obtained. Some very striking results are available on metals of com-

[1] By "glide" is meant the shear strain—the relative movement of two parallel glide planes divided by the perpendicular distance between them. The extension is not a suitable parameter to consider, since it will depend upon the inclinations of the glide planes to the axis in the particular specimen.

paratively high melting-point. Measurements made by Sachs and Weerts[1] and by Osswald[2] show that with a single crystal of nickel a shear strain of 30 per cent. produces an increase of shear strength of fourteen times; with copper a shear strain of 67 per cent. produces an increase of strength of sixty-eight times; while with silver a shear strain of 95 per cent. produces an increase of strength of ninety-two times, all at atmospheric temperature. In many cases the hardening at low temperatures varies as the square root of the glide: at high temperatures it tends to show a linear variation.

Evidence from X-Rays.

We have now to consider how this hardening under strain comes about, and here X-rays furnish a valuable method of attack. If we pass a narrow beam of so-called "white" X-rays, that is, a beam containing a wide continuous range of wavelength, through a fixed single crystal, each set of crystal planes will reflect the rays at a determined angle and we obtain, as is well known, a series of spots, the so-called Laue pattern. If the crystal is turned to another position the spots will turn to fresh positions, just as will a narrow beam of light reflected from a mirror if the mirror is rotated, or similarly, if the beam passes through two crystals oriented in a slightly different way, the spots will appear double. Now if we take a Laue photograph with an unstrained single crystal, we obtain a pattern of single spots, as shown in *Fig. 17*[3], obtained with an unstrained crystal of sodium; but if we strain the crystal the spots are drawn out into smears, called "asterisms", just as would be the case in the presence of a large number of crystals, oriented in slightly different ways, so as to give a train of overlapping spots. *Fig. 18*[3], which is a Laue picture of a crystal of α-iron, extended by about 16 per cent., shows such asterisms. The heavy strain has produced a change in the crystal structure; in fact it looks as if the single crystal is strong because it is no longer a single crystal.

With Miss Chow I carried out some experiments[3] on sodium single crystals, extending them by an equal amount at various temperatures and measuring (1) the stress required to produce this extension, (2) the spread of the asterism. It is easy to measure this with sodium, since the asterism takes the form of discrete spots (see *Fig. 19*)[3], due, as we showed, to recrystallization; the separation of the extreme spots represents the range of rotation of crystalline fragments produced by stress. These experiments show that at low temperatures, where the crystal is strong, the spread is much greater than at higher temperatures, where it is weak, for

[1] *Z. Physik*, 62, 473, 1930.
[2] *Z. Physik*, 83, 55, 1933.
[3] E. N. da C. Andrade and Y. S. Chow, *loc. cit.*

the same strain. We have also been able to establish some numerical regularities. I think, then, that there is now definite evidence that the strengthening is largely due, at any rate, to the break-up of the crystal, to a loss of its single crystal character. Much more experiment is, however, needed to clear up the situation.

Polycrystalline Metals.

This brings us half-way to real metals and the problem which interests the engineer—why are polycrystalline metals strong? There are very few experiments available on the dependence of strength on grain-size. In its simplest aspect this problem demands an absence of plastic deformation, which at once complicates the question, but on the whole it looks as if small grain size is favourable for strength. During plastic deformation slip takes place in the individual crystallites, whose axes are differently oriented, as shown in *Fig. 20* [1], which represents a strained specimen of polycrystalline lead. In some way the boundaries between the crystallites must confer strength, but they cannot do it because they are inherently strong—the number of atoms involved is too small. Expressed in another way the crystal boundaries cannot be regarded as playing the part of the steel rods or expanded metal in reinforced concrete: they are too thin, whatever reasonable strength be attributed to them.

Supposing that, to account for their weakness (low yield-point) as compared with ideal crystals, we accept the hypothesis that the crystallites contain submicroscopic flaws, similar to Griffiths cracks in glass, where the stress-concentration is abnormally high. If a dislocation runs from such a crack it will travel as far as a crystal boundary and will then find, for a region a few atoms thick, at any rate, a place where the regularity necessary for it to propagate itself is lacking—it loses itself, but not because the boundary is hard, rather because it is soft. An experiment which I carried out some time ago may illustrate the point.

To show the effect of crevasses in weakening a solid, I cut transverse slots in celluloid strips, and compared the strength of these strips with that of other unpierced strips, of equal carrying breadth. The weakening effect of the cuts which was to be anticipated did not, however, show itself. Examination shows that at the ends of the crevasse, where the curvature was greatest and tearing should have taken place, the celluloid had yielded plastically, with slight local deformation made evident by a turbid appearance. The theoretical weakening effect of a crevasse is based upon a postulated rigid behaviour of the solid: if the solid yields plastically the local stress is dissipated and the tearing does not take place. The analogy is not a perfect one, but does show how strengthening can take place by weakness. Rather large round holes, cut near the ends of the crevasse,

[1] E. N. da C. Andrade, unpublished.

in which the high local stress would lose itself when the crevasse yielded, might also be used to illustrate the point.

There is no doubt that the intercrystalline boundaries play a very important part in the strength of metals. However, the experimental study of the properties of these boundaries is only just beginning, and, although Chalmers[1] has carried out interesting experiments on the subject, much work with different metals and at different temperatures is required before we have enough material to found a comprehensive theory. The properties of single crystals of metals may seem curious and curiously remote from anything that the engineer experiences, but it is in terms of their behaviour, and of the behaviour of the less regularly ordered atoms in the boundaries between crystals, that we must seek an explanation of the secrets of the ironmaster and of the worker in non-ferrous metals.

Conclusion.

I have tried to indicate to you, then, some of the problems which confront us when we try to explain the simplest features of the mechanical strength of solids, such as the tensile strength, the rupture under shear, work hardening and creep. I have not had time to say much on any of these problems, and if we knew more no doubt my story would have been simpler and more consequential. I have had to lay before you problems which are still in the early stages of attack rather than a compact account of a subject successfully ordered by a comprehensive theory. I would, however, end by a plea that it is a subject well worthy of attention. Even from the industrial point of view the work which I have very tritely and tentatively described is beginning to touch on practice. I have referred to the reinforcing of concrete by glass beams and to hardened glass and glazes, where the state of the surface has had to be considered. I may say also that it has been found necessary to make the tungsten spirals of glow-lamps of single crystal wires, or of wires consisting of very few crystals, for reasons explained by Becker on theoretical grounds, which I have not had time to describe. In a lamp in use the spirals are above the recrystallization temperature, and recrystallization, that is, the transfer of atoms from one lattice to another, is accompanied by marked flow under stress. A single crystal cannot exhibit recrystallization. It is, however, not on these details that I wish to appeal to you.

After the war there will probably be a great movement for the prosecution of research with an immediate practical end in view, and of the importance of such research there is no question, nor will there be any lack of voices shouting in support of its obvious claims. I wish to enlist your sympathy for the type of research which I have been describing, pursued with the more fundamental aim of finding out the mechanism that underlies certain general properties of matter. The work will be undertaken by

[1] Proc. Roy. Soc. A, 175, 100, 1940.

men who long to investigate for learning's sake, lured on by the beauty of the phenomena and the hope of finding a meed of that intellectual, nay spiritual satisfaction which attends the understanding of any small sphere of natural happenings—by men who, varying in ability but not in enthusiasm, desire to be natural philosophers, to use the old and choice phrase. The ends are vast, the approach slow.

The ultimate practical gain may be great, for as we have seen, there are theoretical possibilities of vastly increased strength of materials—where so many of the dreams of inventors are clearly impossible on simple grounds of energy and such-like fundamental considerations, gigantic advances are here, at any rate, not inherently ruled out. Just as the work of J. J. Thomson and his school, carried on without the least thought of practical ends, carried on in the faithful pursuit of scientific truth and beauty, led to the whole modern light electrical industry, so may work of the type which I have glanced at carry in it the seeds of a new metallurgy. I am not suggesting that any of those who have been working in the field are J. J. Thomsons, or even seconds to J. J. Thomson—far from it. I am not appealing to you on the ground of an immediate or ultimate practical gain. I am begging that if the time comes when you are asked whether any shall be allowed to carry on pure experimental research whither the urge leads them, you shall not ask too strictly whether a practical end is in view, but rather, if you think that they are capable and in earnest, are seriously striving in their small way to advance knowledge, that you shall refresh them and wish them God-speed.

Professor C. E. Inglis said that it gave him great pleasure to propose the vote of thanks, though he had to do so with brevity, which he hoped would not be mistaken for lack of appreciation, because with, he thought, everyone else present, he had appreciated fully the admirable Lecture which Professor Andrade had given. He felt that the members had been particularly fortunate in the past in their James Forrest Lectures, and that tradition had been amply fulfilled on that occasion. He considered that one of the many advantages of that annual Lecture was to take members a little out of their groove and to imbue them with a few fundamental ideas. Professor Andrade had done that most admirably, and his Lecture had been of extreme interest; moreover, he had displayed great versatility in the choice of its title, under which almost any branch of science could be brought into the light of mechanical, electrical, and structural engineering. The title had not conveyed any idea of what the Lecture was to be about, so that he had not been able to come prepared with any remarks, and it had been of such interest that he had not been willing to waste time by making notes while the Lecturer was speaking.

The Lecturer had touched upon two extremely interesting subjects; particularly in regard to the strength of materials and solids, which he had shown to be not nearly so high as it ought to be; in fact, under test

it was about a thousand times less than it should be. That aroused considerable curiosity, and opened up wide possibilities for the future. Just as civilization had passed through the stone and brass ages, we were now getting near to the end of the iron and steel age, because iron and steel had the objection that they were crystalline structures, whereas the Lecturer had shown that crystalline structures were, in a sense, things to be avoided! One way of getting away from them would be to develop fundamental strengths which would make them look a back number.

Professor Inglis supported the Lecturer's plea that if, after the war, there should be pressure for applied research, engineers should also give a kindly thought to fundamental research.

Dr. W. H. Glanville, in seconding the motion, said that he had found it very refreshing indeed to hear the Lecture, which was the second of its kind in successive years, and had given considerable food for thought. He agreed with Professor Inglis that the iron and steel age was probably near its end. The Lecturer had furnished much information concerning the fundamentals of those materials and had stated that when such research was more complete it was hoped that even better materials would result. Iron and steel required so much looking after that engineers would really welcome something that could replace them. The Lecturer had observed that engineers were always concerned with things that were too difficult for him; but Dr. Glanville had little hesitation in saying that the subject of the Lecture was much more difficult, even when put in such an able and arresting manner: it was the Lecturer who dealt with fundamental things, and engineers who were concerned with the more simple ones.

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"Stresses Between Tire and Road."

By Alfred Herbert Dorlencourt Markwick, M.Sc., Assoc. M. Inst. C.E., and Herbert James Harold Starks, B.Sc., Ph.D.

TABLE OF CONTENTS.

	PAGE
Introduction	309
Experimental methods	309
Results of normal pressure measurements	311
Results of shear stress measurements	319
Conclusions	322
Acknowledgements	323
Appendix	324

Introduction.

The stresses imposed by the tire of a vehicle upon the surface of a road are of interest from several points of view. In the first place these stresses are responsible for that part of the wear and deformation of the road-surfacing caused by traffic; secondly, the stresses at the surface of contact between tire and road have an important bearing upon the resistance of the tire to skidding; thirdly, tire wear must also depend upon the stresses between tire and road. Research on the measurement of these stresses has been in progress for some time at the Road Research Laboratory of the Department of Scientific and Industrial Research, but the work has been interrupted owing to war conditions. It is therefore considered that the results obtained so far should be rendered available to persons interested in road engineering problems.

The stresses comprise both pressure normal to the surface of the road and shear stresses in the plane of the road; the latter may be resolved into two components, parallel and at right angles respectively to the direction of motion of the tire. Complete information about the stresses imposed by a tire can thus be obtained by measuring three quantities at all points over the surface of contact of the tire.

Experimental Methods.

Previous Work.—Previous experimental work has been confined to measurements of the stresses imposed by a stationary tire upon a plane

surface. The method adopted by H. Martin [1] was to counterbalance with pulleys and weights the forces exerted on a small plunger, the weights being adjusted until dial gauges showed that the plunger had returned to its initial position. Both normal and shear stresses were measured in this way for solid and for pneumatic tires. L. W. Teller and J. A. Buchanan [2] measured the normal pressure between smooth solid and smooth pneumatic tires and a plane surface by measuring the limiting frictional resistance when a small metal bar, flush with the surface of contact, was pulled out horizontally from beneath the tire.

Present Work.—The measurements at the Road Research Laboratory were made under both stationary and moving tires. The static tests were carried out by loading a tire in a testing-machine, whilst the tests with tires of a moving vehicle were carried out under full-scale conditions on a road machine in which a 12-ton lorry travelled at speeds ranging up to 40 miles per hour on a circular track 10 feet wide and 110 feet in mean diameter. A tracking-mechanism permitted the lorry to move radially over the track while the machine was running, rendering it possible to explore the stresses over the whole area of contact of the tire by measurements taken at a fixed point in the track. The duration of the contact between the tire and the road was of the order of $\frac{1}{50}$ second at 40 miles per hour, and it was considered convenient to use electrical methods of recording. The method of converting mechanical stress into an electrical quantity was based upon the fact that stresses applied to a moulded carbon resistor element produce proportionate changes in its electrical resistance. In the static tests, the resistor formed one arm of a balanced Wheatstone bridge; the change in resistance produced by the load applied to the element by the tire upset the balance and produced a small electromotive force which caused deflexion of a galvanometer. In recording stresses under moving tires, the small changes in electromotive force were amplified and made to produce vertical deflexions of an oscillograph-spot which, when simultaneously caused to move horizontally at uniform velocity, recorded on the oscillograph-screen the longitudinal pressure-distribution curve under the tire. A schematic arrangement of the cathode-ray oscillograph equipment, together with a brief description, is given in the Appendix.

Normal and Shear Stress Units.—Several different types of stress unit, all incorporating carbon resistor elements, have been tried, but the latest and most satisfactory designs are those described in the Appendix. These units, which are mounted on a steel plate, can either be used in the testing-machine or laid in the track of the road machine. The same units, therefore, were used for tests on both stationary and moving tires. The carbon

[1] "Pressure-Distribution on the Contact Surface between Tire and Road." *Kraftfahrttechnische Forschungsarbeiten*, Heft 2. Berlin, 1936.

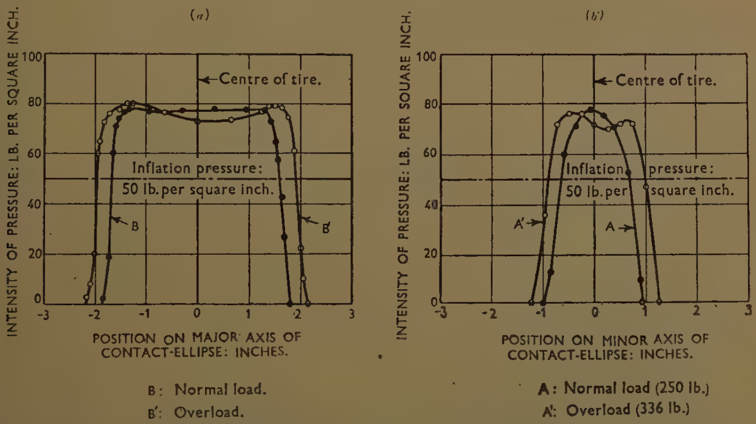
[2] "Determination of Variation on Unit Pressure over the Contact-Area of Tyres." "*Public Roads*," vol. 18 (1937–8), p. 195.

resistor elements used in these stress units were $\frac{1}{8}$ inch in diameter and $\frac{3}{8}$ or $\frac{5}{8}$ inch long, and had a resistance of approximately 50,000 ohms. This size of resistor was chosen after numerous tests, and elements were finally obtained which showed little hysteresis under repeated loading. Care had to be taken to ensure that the elements were at a constant temperature. An initial load of from 5 to 10 lb. was applied and the maximum load was not allowed to exceed 20 lb. The current flowing through the element was of the order of a few milliamperes.

Results of Normal Pressure Measurements.

Normal Pressure on a Smooth Surface. Typical curves for pneumatic tires of various types, showing the normal pressure-distribution on a smooth

Figs. 1.



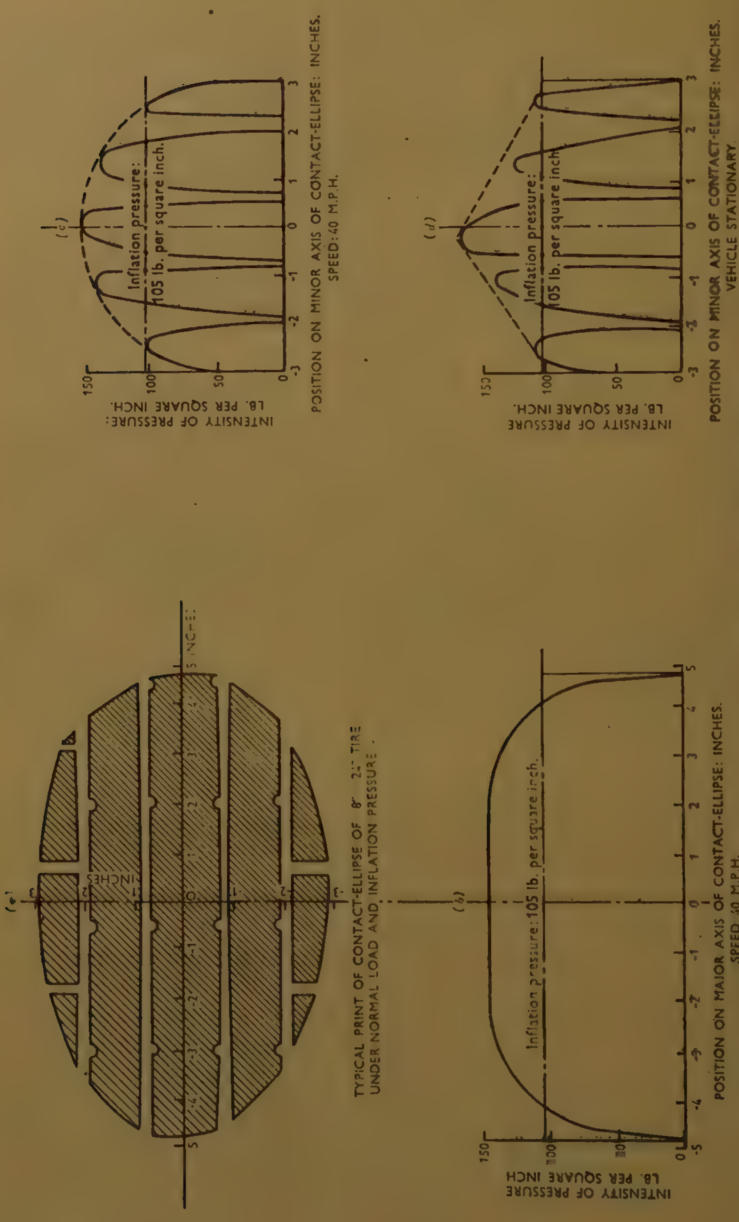
Distribution of Normal Pressure along the Major and Minor Axes of the Contact-Ellipse of a 3-inch × 20-inch Pneumatic Tire (smooth tread).

surface along the axes of the ellipse of contact of the tires, are shown in *Figs. 1* and *2*.

The normal pressure exerted by pneumatic tires on a smooth road is largely controlled by the inflation-pressure. Part of the load, however, is supported by the tire-carcass, and this has the effect of increasing the maximum pressure-intensity to about $1\frac{1}{2}$ times the inflation-pressure. Experiments have shown [1] that the portion of the load carried by the tire-carcass varies considerably with the size of tire: for tires carrying the loads recommended by the manufacturers at normal inflation-pressures, it was about 10 per cent. in the case of an 8-inch by 24-inch tire and 35 per cent. in the case of a 3-inch by 20-inch tire. For a given tire

[1] Report of the Road Research Board for the Year ended 31st March, 1937, pp. 75–77. H.M. Stationery Office, London, 1938.

Figs. 2.



DISTRIBUTION OF NORMAL PRESSURE UNDER 8-INCH × 24-INCH TIRES UNDER AN AXLE LOAD OF 2 TONS.

it varies with the deformation of the carcass, which in turn depends upon the load and the inflation-pressure. Both under-inflation and excessive load can cause undue deformation of the tire-carcass and "overload" the tire.

Figs. 1 illustrate the pressure-distribution under a stationary smooth-treaded 3-inch by 20-inch motor-cycle tire inflated to normal pressure, both when the tire is normally loaded and when it carries overload. A curve similar to A′ (*Figs. 1* (*b*)) was obtained when "overload" was produced by reducing the inflation-pressure. Unduly large deformation of the tire-carcass due to excessive load or under-inflation tends to show itself by the presence of two maxima which occur near the periphery of the area of contact where the flexural deformation of the tire is greatest. Under normal loading conditions, the flexure of the tire-carcass is less severe, and the two maxima merge and give a parabolic pressure-distribution with a maximum at the centre of the tire.

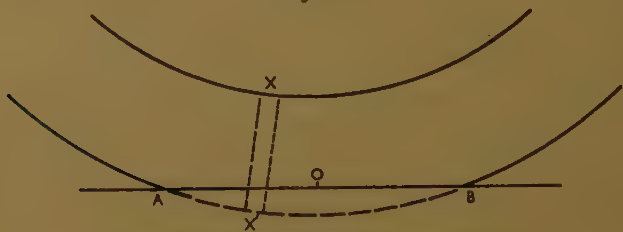
Figs. 2 show the longitudinal and transverse pressure-distributions under an 8-inch by 24-inch treaded pneumatic tire, for which a print of the ellipse of contact is shown in *Figs. 2* (*a*). A typical oscillograph record, obtained when the tire was moving at 40 miles per hour, is shown in *Figs. 2* (*b*). In addition to the increase in mean pressure over the inflation-pressure due to tire-stiffness, an additional increase—amounting in this case to 15–20 per cent.—is caused by the tire tread pattern, since the inflation-pressure pressing over the whole tire-ellipse is transmitted to the road only through that portion of the tread which is in contact with the road. Transverse pressure-distributions under moving and stationary tires are illustrated in *Figs. 2* (*c*) and *2* (*d*) respectively. The curves in *Figs. 2* (*c*) represent the average of a large number of oscillograph records of the type of *Figs. 2* (*b*), at various points on the tire, and are thus very regular. On the other hand, the curve for the stationary tire refers to one particular cross-section, and therefore shows small local irregularities such as those due to uneven wear on the tire-tread. If allowance be made for these small differences, the curves are similar and show that the normal pressure-distribution under the tire is essentially the same whether the tire is moving or stationary. The pressure-distribution curve under each section of the tread, as well as the envelope of the curve as a whole, is parabolic and may be compared with that for a smooth-treaded tire under normal loading conditions (*Figs. 1*).

The effect upon the pressure-distribution curve of irregularities in the tire-tread, such as those due to uneven wear and moulding seams, is pronounced. Uneven wear on a 3-inch by 20-inch motor-cycle tire resulted in an asymmetrical pressure-distribution curve. Moulding irregularities, which occur particularly in new tires, also caused asymmetrical pressure-distributions or pronounced peaks at the centres of the distribution-curves.

The pressure-distribution under smooth solid tires is similar to that

under smooth pneumatic tires, but the normal pressure-intensity is usually much higher. The maximum intensity of pressure was found to be approximately 1·5 times the mean intensity of pressure, in agreement with the measurements of Martin[1]. The pressure-distribution curve approximates closely to a parabola. Martin has suggested that if a solid tire be regarded as composed of a series of independent radial elements, the strains, and hence the normal stresses imposed upon the road, will increase from zero at the edges of contact proportionately to the height of the segment AB (*Fig. 3*). This segment, therefore, represents the distribution of normal pressure beneath a solid rubber tire, and the distribution-curve will be approximately parabolic.

Fig. 3.



Deformation of a Solid Rubber Tire.

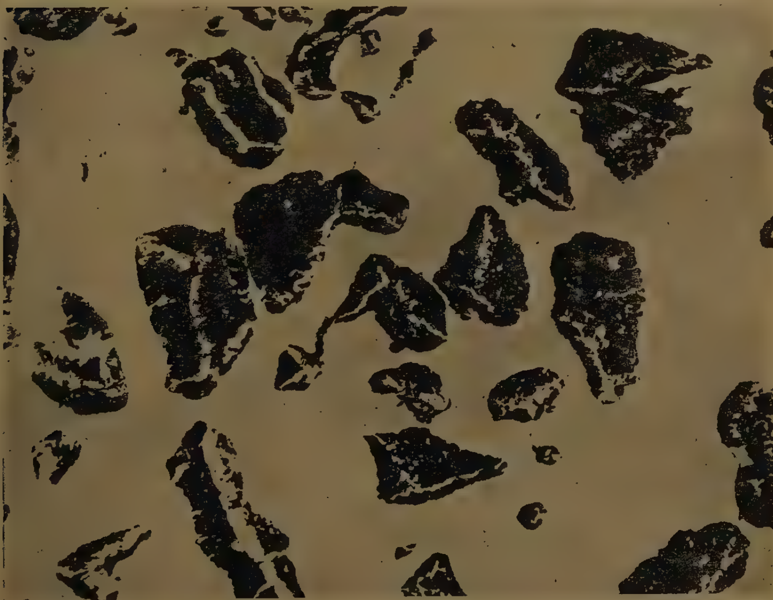
Normal Pressure on a Rough Surface. The results described are for a smooth road, which was represented experimentally by a polished steel plate. In practice, smooth road-surfaces seldom occur, but the surfaces consist of numerous small projections from a smooth surface. Some indication of the pressure on such projections may be obtained by regarding the tire as a semi-infinite elastic solid which is deformed by a rigid die projecting into it. Various shapes of cylindrical projections for which mathematical solutions exist were illustrated in the Report of the Road Research Board for 1938[2]. This work demonstrated that very high intensities of pressure exist on sharp edges. This deduction has been confirmed by texture prints of road-surfaces made by inking the surface and taking off an impression with a rubber roller. The pressure on sharp edges is frequently sufficient to expel the ink, as illustrated in *Fig. 4*. Here the ink has been expelled along the white lines, which represent a series of ridges in the surface. Since the projections are ridges, a two-dimensional solution is more appropriate than the assumption of cylindrical projections. No mathematical solution of the two-dimensional problem has hitherto existed, but the late Professor A. E. H. Love, F.R.S., communicated to the Laboratory a general solution of this problem, together with solutions of a number of special cases.

[1] See footnote 1, p. 310, *ante*.

[2] Report of the Road Research Board for the Year ended 31st March, 1938, p. 162. H.M. Stationery Office, London, 1938.

In the case of a projecting cylinder, the magnitude of the stresses depends upon the total projection of the cylinder below the original surface of the elastic solid. In the two-dimensional case of a projecting ridge, however, the stresses were found to depend upon the height of that portion of the ridge in contact with the elastic body, which, on account of the depression produced in the surface of the body, is correspondingly less than the total projection. In both cases the expressions for the stress

Fig. 4.



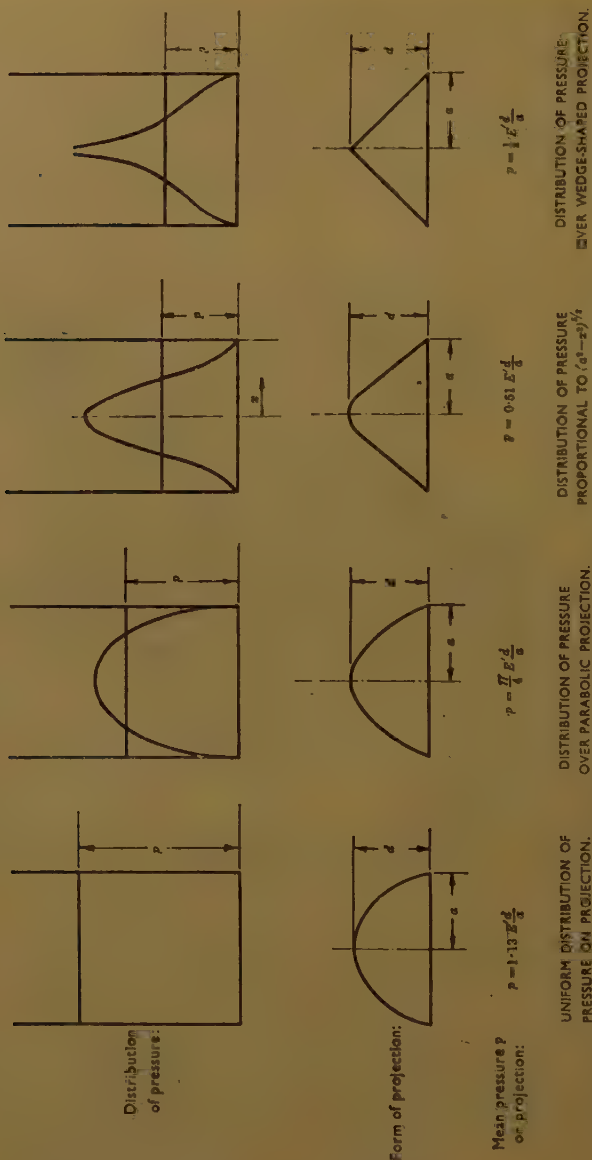
TEXTURE PRINT OF ROAD SURFACE. (TWICE FULL-SIZE.)

involve the ratio height/width, and thus depend upon the shape of the projection and not upon its actual size.

A number of special two-dimensional cases are illustrated in *Figs. 5.* These indicate that the sharper the projection the larger are the maximum stresses that occur. For sharp projections the mean stress on the surface in contact with the tire will be small in relation to the pressures on the ridges which were sufficient to expel the ink in the texture print illustrated in *Fig. 4.* In the formulas given in *Figs. 5*, $E' = E/(1 - \sigma^2)$, where E denotes the modulus of elasticity and σ denotes Poisson's ratio for the tire-tread material.

Some experimental justification for applying this theory to the case of a tire has been obtained by causing the plunger of the normal pressure-unit (*Fig. 8*) to project by varying amounts. The experimental results

Figs. 5.



Two-Dimensional Surface Irregularities over which the Pressure-Distribution can be Calculated.

were compared with the mathematical solution of the projection of a flat-ended cylindrical die given by Timoshenko[1]. The mean pressure p

[1] S. Timoshenko, "Theory of Elasticity," pp. 334 and 339. McGraw-Hill Book Co., New York and London, 1934.

exerted on a cylindrical plunger of radius a projecting by an amount z into a semi-infinite elastic solid is given by the formula :—

$$p = \frac{2E}{\pi(1-\sigma^2)} \cdot \frac{z}{a} = \frac{2E'}{\pi}\frac{z}{a}$$

where E, σ, and E' are defined as indicated above.

Tests in which the ratio $\frac{z}{a}$ of a cylindrical projection was progressively increased, other conditions being maintained constant, have shown that the pressure on the top of the projection increases proportionately with $\frac{z}{a}$ when $\frac{z}{a}$ is small (less than about unity). When $\frac{z}{a}$ becomes larger the rate of increase of pressure with increase of $\frac{z}{a}$ changes sharply to about half its former rate. This may be due to the fact that, when the projection-ratio $\frac{z}{a}$ is small, the tire-tread is not in contact with the sides of the projection, but that, when the projection-ratio is large, the tread makes contact over a large portion of the sides of the projection. It is therefore possible that, above a certain critical projection-ratio, the pressure exerted on the top of the plunger may be partly relieved by the rubber pressing on the sides of the plunger.

Tests have also shown that the modulus of elasticity, determined from the increase in pressure, is simply related to the hardness of the rubber, measured by the Shore "Durometer" or the Dunlop "Hardness-gauge." The increase in pressure on unit projection ("projection hardness value") is, however, much more susceptible to small changes in tread-hardness than are these measuring instruments. The effect of small differences of temperature and of temperature-gradients through the tire-tread may be readily observed, for these affect the modulus of elasticity, and hence the hardness of the rubber. Considerable temperature-gradients have been observed as the tires of the road machine lorry heat up under continued running. Wide variations have also been observed between similar tires under identical conditions of load and inflation-pressure. In some tread-materials the temperature-gradient is much less than in others and the causes are not yet fully understood by rubber technologists; even tires of the same mix are found to be subject to considerable variations.

In spite of the complexities introduced by the temperature-gradient in the tire tread, the following conclusions of the theory have been found to apply :—

(1) The mean pressure on a given projection depends upon the hardness of the tread rubber.

(2) For projections of given overall dimensions, the mean pressure exerted depends upon the shape of the projection.

(3) Exceedingly high local concentrations of pressure occur on sharp projections.

These conclusions have an important bearing upon road behaviour. For example, the progressive increase in smoothness of a bituminous surfacing with wear probably arises from the high downward thrusts exerted by a tire on the projections that constitute the surface texture of a road. Not only are stones pushed into the matrix, but also sharp edges on the aggregate are worn down by the high local stress-concentrations that exist on the peaks of the roughnesses.

The effect of tread-hardness upon road friction is also partly explained by the theory. Measurements of sideway force coefficients have been made with the Road Research Laboratory's motor-cycle equipment under carefully-controlled conditions, with tires of hardness ranging from 33 to 84 on the Shore scale. Typical values of the coefficients obtained at 30 miles per hour are given in Table I [1].

TABLE I.—VALUES OF SIDEWAY FORCE COEFFICIENT ON THE KINGSTON BY-PASS, SECTION 4, 17 MAY, 1938. (ROAD WET.)

Shore hardness of tire-tread.	Sideway force coefficient at 30 m.p.h.
78	0·78
59	0·63
48	0·36
44	0·31
33	0·24

It will be seen that the hardness of the tire-tread has considerable influence upon the frictional properties. This is to be expected from the theory, since the pressures on the projections forming the surface texture of the road are much higher when the tread rubber is hard than when it is soft, and the film of moisture tending to promote slipperiness will therefore be expelled more readily. The range of hardness covered in Table I is, however, much wider than would be experienced in practice, wherein the usual range is from 50 to 70.

The size and shape of the projections forming the surface texture of a road will similarly affect the pressures exerted by the tire on these projections. Surface texture will thus be one of the principal factors governing road friction. The numerous measurements of sideway force coefficient made by Bird and Scott [2] led them also to this conclusion.

[1] See also Report of the Road Research Board for the year ended 31st March, 1939, p. 129. H.M. Stationery Office, London, 1939.

[2] G. Bird and W. J. O. Scott, "Studies in Road Friction. I. Road Surface Resistance to Skidding." Road Research Technical Paper No. 1. H.M. Stationery Office, London, 1936.

Results of Shear Stress Measurements.

Shear Stresses on a Smooth Road. In addition to normal stresses, horizontal shear stresses are set up over the contact-area of the tire both when the tire is at rest and when it is moving over the surface of the road. These stresses arise because of the resistance offered by the road-surface to lateral displacement of the tread. Under a stationary tire the shear stresses must, by symmetry, be zero at the centre of the tire, and in order to satisfy the conditions of equilibrium they must be of opposite sign on either side of the centre.

The direction and approximate distribution of the shear stresses under a solid tire may be deduced from *Fig. 3.* Each of the elements XX′ will tend to expand laterally under compression and move away from the centre of the contact ellipse. This tendency is resisted by tensile stresses in the road-surfacing. The shear stresses will therefore be directed outwards and will increase from zero at the centre O. This has been confirmed experimentally by Martin [1].

The conditions under a pneumatic tire are different. Compression of the tread will tend, as before, to produce tension in the road-surface, but the effect is more than offset by stresses due to flexure of the tire-carcass over the area of contact of the tire. This flexure will tend to produce tension in the inner portion and compression in the outer portion of the tire-carcass. Since the tread is not free to move where it is in contact with the road, compressive stresses which would otherwise occur entirely in the tread-rubber are also induced in the road-surfacing. In addition, the relief of the circumferential tensile stresses present in the tire-carcass will also tend to produce compression of the road-surface; but measurements of the extensibility of the carcass of an 8-inch by 24-inch tire suggest that this factor is relatively unimportant.

Measurements of shear stress on stationary pneumatic tires have been made with the apparatus described in the Appendix. *Fig. 6* shows the shear stress measured under a stationary 3-inch by 20-inch motor-cycle tire along the minor axis of the ellipse of contact on a plunger $\frac{1}{8}$ inch in diameter projecting 0·01 inch. The stresses are directed inwards, and it is evident that the shear stress is zero at the centre and at the periphery of the ellipse of contact and that the areas under the positive and negative parts of the curve are approximately equal, that is, there is no resultant force along the minor axis. This also is in agreement with the results of Martin [1].

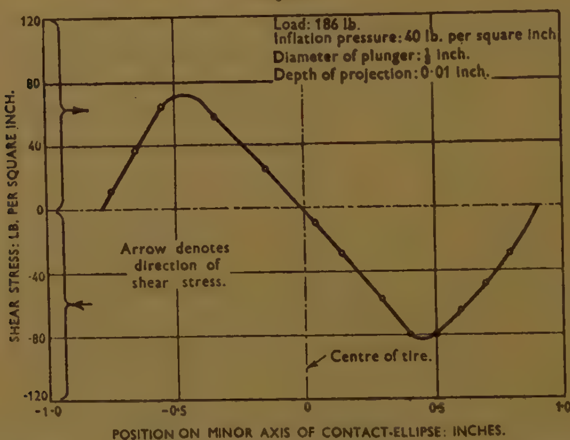
Under a moving tire, however, the stress-distribution is no longer symmetrical, since there will be a net resultant force in one direction equal to the tractive effort, and this force will act in opposite directions under driving and under driven wheels.

---

[1] *Loc. cit.*

The shear stresses which occur under a moving pneumatic tire have been studied on the road machine. This portion of the work is incomplete and the results so far obtained are of only a preliminary nature. Typical oscillograph records of the shear stress distribution measured in the direction of motion under the outer tire of a pair of rear driving-wheels of the road machine vehicle are reproduced in *Figs.* 7 (*a*) and (*b*), for dry and wet road conditions respectively. When the road is dry, alternating stresses of relatively high frequency occur in the second half of the record. The general appearance of this portion of the record

Fig. 6.

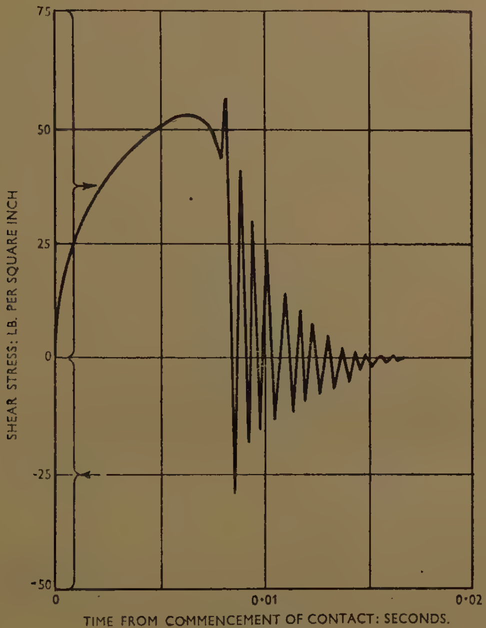


Distribution of Shear Stress along Minor Axis of Contact Ellipse under a Stationary 3-inch × 20-inch Smooth Tire.

suggests that some natural frequency, either of the shear stress unit or of the tire, is excited under dry running conditions. The frequency observed is approximately 1,400 cycles per second. Alterations in the design of the shear stress unit and in the dimensions of the carbon resistor elements caused little difference in the shape of the record, and it appears unlikely that the vibration corresponds to a natural frequency of the shear stress unit.

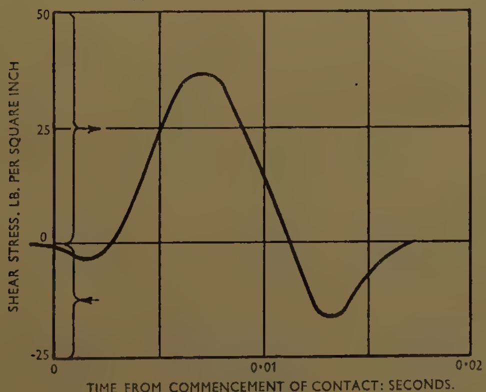
These records, however, refer to a tire moving in a circular track of comparatively small radius, so that the shear stress distribution under the tire may not be similar to that under a tire moving in a truly linear path. For instance, the tire-tread showed definite signs of "tire scrub," due probably to the circular motion coupled with the radial tracking motion. The squeak of tires produced when a vehicle is braked hard shows that vibrations of relatively high frequency can occur under tires. Moreover, examination of certain skid-marks on roads has revealed clearly the existence of periodic markings which may be due to the same cause.

Figs. 7.



(a) Dry road

Arrow denotes direction of shear stress.



(b) Wet road

Load: 2 tons.

Tire size: 8″×24″

Diameter of plunger: ⅛ inch (flush).

Inflation pressure: 105 lb. per square inch.

Distribution of Shear Stress at a Given Point on the Track under the Driving-Wheel Tire of a Vehicle moving at 40 m.p.h. in a Circular Path.

Further work is required before a complete explanation of *Fig*. 7 (*a*) can be given.

When the road is thoroughly wet, stress-alternations do not occur (*Fig*. 7 (*b*)), but when the road is partly wet, the shape of the record obtained is intermediate between that of *Fig*. 7 (*a*) and that of *Fig*. 7 (*b*), the transition taking place gradually as the road dries or becomes wet.

The area under the curves in *Figs*. 7 is proportional to the shear force exerted on the road by a narrow strip of the tire lying along the major axis of the area of contact. Areas above the axis represent a resultant force in a backward direction on the road, and those below the axis a resultant force in the opposite direction. Thus in *Figs*. 7 there is a net resultant backward force on the road corresponding to the tractive effort.

Approximately uniform acceleration and deceleration (of the order of 2 feet per second per second) did not alter the maximum value of the shear stress, but the general stress-distribution changed in the manner which would be expected in order to give the required tractive effort.

The shear stress distribution under the tire measured in a direction at right angles to the direction of motion is somewhat similar to that shown in *Figs*. 7, and exhibits similar characteristics under wet and dry road conditions.

Shear Stresses on a Rough Surface. A few preliminary observations have shown that the shear stresses on small projections from a smooth surface are greater than on the smooth surface itself.

Conclusions.

I. The tires of vehicles exert the following stresses on the road :—

(*a*) *Normal Stresses.*

(1) Pneumatic tires exert maximum normal stresses proportional to, and approximately 1½ times, the inflation-pressure. The maximum pressure will be of the order of 150 lb. per square inch and will occur only under the tires of heavy lorries.

(2) Solid tires exert normal pressures higher than those found under pneumatic tires.

(3) The normal stresses appear to be independent of speed, and are the same for moving and for stationary tires.

(4) Very high local intensities of normal pressure occur on sharp projections in the road-surface. These pressures depend upon the hardness of the tread-rubber and upon the shape of the projection : they are practically independent of tire-size and inflation-pressure.

(*b*) *Shear Stresses.*

(1) Under pneumatic tires the shear stresses are directed inwards and give rise to horizontal compressive stresses in the road-

surface beneath the tire. They may attain 50 lb. per square inch.

(2) Under solid tires the shear stresses are directed outwards and give rise to horizontal tensile stresses in the road-surface beneath the tire.

(3) The shear stresses exerted by a pneumatic tire running on a circular track depend upon the wetness or dryness of the road. Under dry road conditions rapid alternations in stress occur as the portion of the tire in contact begins to leave the road. On a wet road these alternations do not occur.

(4) The distribution of shear stresses under tires is influenced considerably by the magnitude and direction of the resultant thrust of the wheel on the road.

II. The large downward forces existing on projections from the road-surface are probably mainly responsible for the progressive increase in smoothness with wear that is characteristic of bituminous surfacings.

III. The hardness of the tread-rubber has a considerable influence upon the resistance of a tire to skidding.

IV. The difference in the nature of the shear stresses, and in the resultant horizontal stresses in the road-surface, is believed to be the essential difference between pneumatic and solid tires from the point of view of road wear.

Acknowledgements.

The work described was carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. Miss E. B. Philip, M.A., obtained special solutions of the mathematical problem, and Mr. G. Bird, B.Sc., A.M.I.Mech.E., A.M.I.A.E. was responsible for the sideway force coefficient measurements recorded in Table I, and assisted with road friction problems generally. The Authors also desire to record their indebtedness to the late Professor A. E. H. Love, F.R.S., for the solution of the two-dimensional problems and for the interest he took generally in the mathematical side of the work.

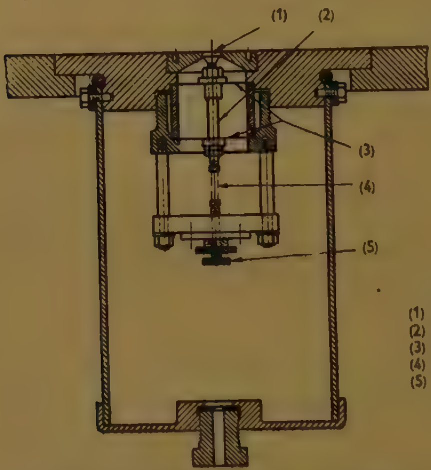
The Communication is accompanied by nine sheets of drawings and by one photograph, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Description of Apparatus.

Normal-Pressure Unit.—The normal-pressure unit is illustrated in *Fig. 8.* The tire bears on the plunger-cap (1); caps of various external diameters are available. The load is transmitted through a plunger (2) mounted on two phosphor-bronze diaphragms (3), which guide it and serve also to keep moisture from the interior

Fig. 8.

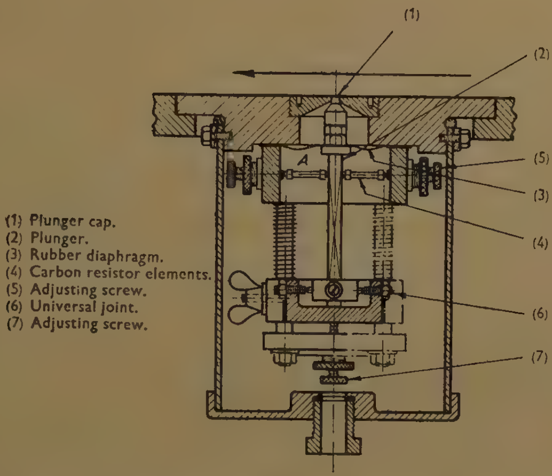


Normal-Pressure Unit for Measuring the Normal Stresses under the Tire.

of the unit. The lower end of the plunger bears on the resistor element (4), the metallized ends of which are fitted into spherical-ended metal caps. The whole is carried by the adjusting screw (5), which permits the level of the top of the plunger-cap above the road surface to be varied at will. The screw itself is carried by an ebonite plate to insulate the lower end of the resistor from the frame of the unit. A metal case encloses the whole unit, protecting it from moisture and from electrical interference. Screened leads are taken from the carbon element to the oscillograph through a watertight screw cap in the base of the cover.

Shear Unit.—The shear unit is illustrated in *Fig. 9.* The tire bears on the plunger-cap (1); caps of various external diameters are available. The clearance space between the plunger and the surrounding ring is packed with thick grease. The horizontal shear force applied by the tire is transmitted through a plunger (2) passing through a rubber diaphragm (3) which serves to keep moisture from the interior of the unit. The plunger is maintained in position by four carbon resistor elements (4), arranged in pairs at right angles. These elements, which are held between the plunger and the adjusting screws (5), are used to measure the forces applied. The plunger is pivoted at its lower extremity on a universal joint (6). The level of the plunger-cap relative to the surface of the plate may be varied by means of the adjusting screw (7). The whole unit is enclosed in a metal case to protect it from moisture and from

Fig. 9.



SHEAR UNIT FOR MEASURING THE SHEAR STRESSES UNDER THE TIRE.

electrical interference. Screened leads are taken from each of the carbon elements to the oscillograph through a watertight screw-cap in the base of the cover.

Cathode-Ray Oscillograph Equipment.—Fig. 10 is a block diagram of the various units of the cathode-ray oscillograph equipment. The changes in resistance of the

Fig. 10.

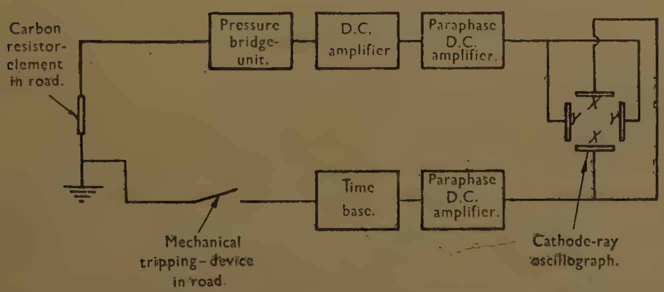


DIAGRAM OF CATHODE-RAY OSCILLOGRAPH EQUIPMENT FOR MEASURING NORMAL AND SHEAR STRESSES UNDER THE TIRE OF A MOVING VEHICLE.

carbon element caused by alterations in stress are converted into changes in electromotive force by means of a bridge unit. These small changes in electromotive force are amplified and applied to the Y-plates of a high-vacuum cathode-ray oscillograph, where they cause vertical movements of the oscillograph spot. Horizontal movement of the spot is obtained by means of a single-stroke time-base, the output of which is amplified and applied to the X-plates of the oscillograph. The time-base is actuated by mechanical contacts which are closed by the tire of the lorry. The resultant motion of the spot traces the stress-distribution curve under the tire.

Paper No. 5238.

"The Gebel Aulia Dam."

By ALEC GEORGE VAUGHAN-LEE, M. Inst. C.E.

(Ordered by the Council to be published with written discussion[1].)

TABLE OF CONTENTS.

	PAGE
Historical	326
Short description of the works and site	332
Materials for the works	333
Solid dam	335
Sluice dam	336
Training walls and aprons	337
Lock guide-walls and fish-ladder	337
Embankment dam	340
Steelwork and mechanical equipment	341
Colony	346
Execution of works	346
Operation of reservoir	347
Conclusion	348

HISTORICAL.

SIR WILLIAM GARSTIN, G.C.M.G., Under Secretary of State for Egyptian Public Works, after travelling practically the whole course of the White Nile and the Semliki river from Victoria Nyanza to Khartoum, submitted, in March 1904, a report[2] to Lord Cromer on the basin of the Upper Nile. He advocated that the waters of the Blue Nile should be used for the benefit of the Sudan, and the waters of the White Nile for the benefit of Egypt. He proposed cutting a channel through the sudd region in the Southern Sudan to produce a better run-off of the waters of the White Nile and to obviate the losses by evaporation in the swamps, or sudd region, south of Malakal. This would have provided more summer-water for Egypt, but it was considered too costly. Even to-day, although large sums of money have been spent by the Egyptian government in surveying and investigating the different proposals for this sudd channel, no final decision has been made. The information on the hydraulic data and general conditions in the upper White Nile are not yet such as to justify

[1] Correspondence on this Paper can be accepted until the 15th August, 1941, and will be published in the Institution Journal for October, 1941.

[2] "Report upon the Basin of the Upper Nile." National Printing Department, Cairo, 1904.

the commencement of such a huge undertaking. Moreover, the financial aspect of the proposal must be considered.

Sir William Garstin also advocated the construction of a dam at Lake Albert to raise the water-level of that lake, but unless the channel through the sudd region is formed to facilitate the flow of water down to Khartoum, the construction of the dam on Lake Albert should be deferred until the two schemes can be put in hand at approximately the same time.

In order to obtain more summer-water in Egypt from the White Nile, a proposal was put forward, about 1910 or 1911, to construct a dam on the White Nile, and Mr. P. M. Tottenham, C.M.G. (then Inspector-General of Irrigation for Egypt in Sudan), in his report on White Nile projects, in 1913, proposed to construct a rubble masonry barrage at Gebel Aulia, situated in the low-water channel of the river. The reservoir level would have been at +377·68 metres above mean sea-level at Alexandria, and a navigation lock was to be provided.

Gebel Aulia is on the White Nile 42 kilometres (26 miles) above Khartoum, where there is a sandstone *gebel*, or hill, on the east bank of the river, rising 200 feet above the river-level. The hill is about 1 kilometre in length by 300 metres in width, and is an outstanding landmark in the district.

Investigations had been carried out during 1912 and 1913 as to whether there were other possible sites for a dam on the White Nile, but Gebel Aulia was found to be the most suitable. During 1914 and 1915 borings were made at this site, but it was not until January 1917 that Gebel Aulia was definitely selected, and preliminary drawings were prepared in May 1917 by the Irrigation Department of the Ministry of Public Works in Cairo. This proposal was to construct a gravity-dam of sandstone obtained from the *gebel* on the east bank of the river, and to build the structure to such a height as to impound water to +380·00 metres above datum. This masonry gravity-dam was to have extended right across the valley for 5 kilometres, and to have contained forty sluice-gates, each 4·5 metres high by 3·0 metres wide. If this proposed water-level was to be maintained, an additional dam about 1,700 metres in length would have been necessary on the east side of the *gebel*, where the land is low.

This dam was to have served two purposes: (*a*) the protection of Egypt from very high flood, and (*b*) the impounding of water against a period of shortage of water. As regards using this dam as a means of flood prevention, the effects of the flood in the Blue Nile at Khartoum have to be considered, as this causes a rise of water-level in the valley of the White Nile by ponding up the water of the latter for about 350 kilometres during a high flood, although the discharge of the river is seldom entirely stopped. When the flood in the Blue Nile begins to subside, the ponded-up waters in the White Nile valley are released automatically and pass down the river. It is at this stage that such a high-level dam would be useful to postpone the discharge of the impounded water until a more

22

opportune time. Generally speaking, the maximum height of the flood at Khartoum depends almost entirely on the discharge of the Blue Nile, which at the period in question is approximately six or seven times the discharge of the White Nile. However, once the crest of the flood in the Blue Nile had passed Khartoum, the sluice-gates in the Gebel Aulia dam being closed, the water impounded in the valley of the White Nile would be prevented from following the flood-water of the Blue Nile into Egypt, thus reducing the level of the flood in the main Nile and lessening the risk of burst Nile banks in upper and lower Egypt, with consequent damage and probable loss of life by floods. The water impounded behind the Gebel Aulia dam could be released at any time to replenish the Aswan reservoir as the demand for water in Egypt arose, and thus the second function of a storage reservoir would be fulfilled.

In 1917, Sir William Garstin and Sir Arthur Webb reported favourably on the scheme for a high-level reservoir at Gebel Aulia, and in the financial year 1917–18 a credit of £E15,000 * was granted by the Egyptian Government for preliminary expenses in connexion with the project. Boring operations and the sinking of trial-pits were continued during 1917, and in August of that year a commencement was made with the construction of the housing accommodation for the supervisory staff and a detail survey was made of the site.

On the 4th February, 1918, excavation for the foundations of the dam was started on the east bank of the river, but was terminated at the end of the month.

In January 1919, a dipper dredger worked experimentally on a cut on the west bank of the low-river channel, and excavated about 25,000 cubic metres on the line of the dam. The detail surveys of the reservoir area were continued during 1919, and precise levelling was continued up the river as far as Malakal. Borings were also continued on the proposed line of the dam. The native village at Gebel Aulia was expropriated and rebuilt on another site. Further experimental excavation by the dipper dredger was continued during the latter part of 1919. During this year, the Sudan Construction Company had undertaken to carry out the construction of the dam for the Egyptian government, and in December 1919, the first batch of Egyptian labourers arrived and opened out sandstone quarries at the *gebel* on the east bank, and at another quarry on the west bank of the river. In all, about 22,000 cubic metres of rubble stone were produced between December 1919 and March 1920.

In March 1920, work on the Gebel Aulia dam was suspended, pending the publication of the report of the Nile Projects Commissions, the members of which were Messrs. Gebbie, Cory, and Simpson. They were appointed in January 1920, to examine into and report, *inter alia*, upon the projects prepared by the Ministry of Public Works for the better regulation of

* £E1 = £1 0*s*. 6·15*d*.

the river Nile for the supply of water for the benefit of Egypt and the Sudan. The Gebel Aulia dam was one of these projects. The Report [1] of this Commission recommended that the construction of the dam should be undertaken forthwith. In August 1920, work was slowly resumed on the dam, but was much hampered by lack of labour and transport facilities. Excavation for the foundations of the dam had been proceeded with on both banks of the river, but no masonry work was ever built.

In 1921, the programme of work on the dam was greatly curtailed on account of financial stringency in Egypt. The work at the site was confined, therefore, to the completion of the houses in the Government and contractors' colonies, and the installation of workshops, water-supply, electric light, and other services.

On account of the general rise in prices of materials, plant, and labour, which had taken place since the war, it was now found that the dam at Gebel Aulia could not be carried out for the original estimate of £E2,500,000, the revised estimate being £E6,500,000; and by an order of the Council of Ministers, dated the 25th May, 1921, the works were shut down completely. The staff of the Sudan Construction Company did not finally depart until February 1922, having handed over the vast quantities of stores, plant, materials, etc., that had been accumulated. The total expenditure that had been incurred on the Gebel Aulia project up to the 31st March, 1922, amounted to £E981,503.

In November 1921, Mr. C. E. Dupuis[2], late adviser to the Ministry of Public Works, had been instructed by the Council of Ministers to report on the position and activities of the State Service of Irrigation of the Egyptian Government, with special reference to its relations with other Government services, and the best program of work for the agricultural development of Egypt. Among the many subjects dealt with by Mr. Dupuis was the Gebel Aulia dam as a means of supplying Egypt with more water for irrigation during the summer. He advocated a dam impounding water to a level of +377·20 metres above datum instead of +380·00 metres, thus abandoning the idea of using the high-level reservoir as a means of flood-protection in Egypt. He also suggested the possibility of reducing the cost of the project by modifying the height of the dam and introducing economies in the design, and he proposed that the construction of the dam should be let to a substantial firm of contractors under a properly drawn-up contract.

Towards the end of 1923, the study of the smaller project suggested by Mr. Dupuis was commenced. This consisted of a sandstone gravity-dam across the low-river channel, of a length of about 1,700 metres with an embankment to the westward thereof.

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1 "Report of the Nile Projects Commission." Government Press, Cairo, 1920.

2 "Report on the position and activities of the State Service of Irrigation in the Egyptian Government", June 1923. Government Press, Cairo, 1925.

In 1924, when the late Sir Maurice Fitzmaurice, C.M.G., Past-President of The Institution, was in the Sudan inspecting the Sennar dam, then in course of construction, the Minister of Public Works, His Excellency, Sir Ismail Sirry Pasha, K.C.M.G., requested him to report on the Gebel Aulia dam, with the following terms of reference: "To examine the site of the Gebel Aulia dam, give an opinion on the alternative designs and methods of construction proposed, and suggest any further information required in order that the best possible design may be selected."

Accordingly, Sir Maurice Fitzmaurice and the Author visited Gebel Aulia in February 1924, and inspected the site, and in a report, dated the 26th March, 1924, Sir Maurice recommended that the dam should be constructed to impound water to a level of +377·20 metres above datum. The eastern portion of the dam should be a gravity masonry dam founded on the rock, and westwards of the masonry dam for a length of about 3·2 kilometres (about 2 miles), an embankment dam should be thrown up on the existing surface of the ground, and two rows of interlocking mild-steel sheet-piling should be driven to the rock-level as a cut-off, and the tops of these piles should be 2 metres above reservoir-level. The slopes of the embankment should be pitched with stone, and a wave-breaker of heavy rubble should be provided on the upstream face of the embankment dam. Sir Maurice expressed the opinion that the gravity masonry dam should not be constructed of sandstone, as he deemed that this local material was unsuitable on account of its variable quality, lightness, softness, and porosity, but that granite should be used instead of sandstone. He also recommended that granite-faced aprons should be provided immediately downstream of the sluices.

Following this report, Mr. W. D. Roberts, Inspector-General of Irrigation in the Sudan for the Egyptian Government, prepared preliminary designs for the dam based on Sir Maurice's proposals, and obtained further information with regard to granite and sand of suitable quality. More borings and wells were put down and experiments were initiated over the reservoir area, to ascertain the absorption properties of the soil forming the bed of the reservoir.

In August 1925, the Author's firm were appointed by the Egyptian Government to be consulting engineers for the construction of the Gebel Aulia dam in the Sudan, and the Nag Hammadi barrage in upper Egypt[1]. They were instructed to prepare plans and contracts for both these works, the intention being that a contract should be let as soon as possible for the Gebel Aulia dam, and that the construction of the Nag Hammadi barrage should commence about a year later. The designs for the Gebel Aulia dam followed the recommendations of Sir Maurice Fitzmaurice, who had died in the autumn of 1924. Forty-six sluices were provided

[1] A. R. Ellison, "The Nag Hammadi Barrage." Minutes of Proceedings Inst. C.E., vol. 232 (1930–31, Part 2), p. 340.

for the discharge of the river, and a navigation-lock 80 metres long by 14 metres wide was incorporated in the design. The roadway-level of the dam was fixed at +380·00 metres and the sill-level of the sluices at +369·00 metres above the datum of mean sea-level at Alexandria. In May 1926, tenders were invited for the construction of the Gebel Aulia dam.

Before tenders could be obtained, a change of Government took place, and the proposal to construct the Gebel Aulia dam was again abandoned, chiefly for political reasons, although it was decided to proceed with the construction of the Nag Hammadi barrage.

In May 1929, the project for constructing a dam at Gebel Aulia was revived, and the consulting engineers received instructions to redesign the dam, with eighty sluice openings, each 4·5 metres by 3·0 metres, thirty to be blocked with reinforced diaphragms and thus to be available for future use should hydraulic developments on the upper White Nile occur. The roadway-level and the sill-level of the sluice-gates were maintained at +380·00 and +369·00 metres respectively, and the lock was widened to 18 metres to allow the passage of large dredgers to be used in the sudd region. The design of the masonry dam was prepared to provide sufficient strength and stability to allow, without any thickening of the structure, such subsequent heightening as would enable water to be impounded to the level of +380·00; although in the first instance the water-level was to be +377·20. The design of the embankment portion of the dam was improved by the introduction of a concrete-in-mass core-wall, 3 metres in thickness between the lines of steel sheet-piling, this core-wall being founded on the rock.

In the summer of 1929, British contractors visited the site at low-river to obtain information for tendering, but the Egyptian Government again decided not to proceed with the work that year and instructions were received to postpone indefinitely the issue of invitations to tender.

In December 1931, the project was again revived, and in the following session of the Egyptian Parliament, an Act was passed, and a Royal Decree was promulgated on the 19th July, 1932, ordering the immediate construction of the Gebel Aulia dam. In July 1932, the consulting engineers received instructions from the Minister of Public Works to revise the design of the dam again. The modifications consisted in lowering the roadway-level 1·00 metre to +379·00 metres, whilst sixty sluice openings were to be provided instead of eighty, ten of these being blocked up temporarily, and available for future use. The level of the sills of the sluice-openings was also lowered 1·00 metre to +368·00 metres. The lowering of the sill of the sluice-openings was an improvement, as it was considered easier to calibrate the discharge, since the openings under the gates would then generally be totally submerged.

Tenders were invited from seven selected firms of British contractors, four of whom submitted tenders. On the 25th June, 1933, the contract was awarded to Mr. J. W. Gibson, who, having formed a company with

Messrs. Pauling & Co., Ltd., under the title of Messrs. Gibson & Pauling (Foreign), Ltd., was instructed by the Egyptian Government to commence work immediately. The formal contract was signed in Cairo on the 4th September, 1933.

It will thus be seen that the project for the construction of the Gebel Aulia dam, since its inception to the date of letting a contract in 1933, has passed through many vicissitudes, not unmixed with finance, politics, and alterations of designs. The political aspect was mostly on account of the objections raised to constructing a dam in the Sudan with Egyptian money, notwithstanding the fact that the impounded water was solely for the use of Egypt. There was also a feeling prevalent in Egypt in some quarters that sabotage at the dam might interfere with or damage Egyptian agriculture, but this could occur even at the famous Aswan dam in Egypt.

Short Description of the Works and Site.

The dam is probably one of the longest in the world for impounding a river, the reason being that the valley of the White Nile is a shallow trough approximately 4 to 8 kilometres in width, situated in a flat arid plain, with an occasional *gebel* or small hill showing up as a landmark. The actual width of the valley at impounded water-level at the site of the dam is 4·7 kilometres, which necessitated a dam of 5 kilometres in length, as the rise of the ground at the west end is very gradual.

The dam is divided into two sections, namely, the masonry dam and the embankment dam (Figs. 1 and 2, Plate 1), the former being subdivided into various sections. Table I gives the lengths :—

Table I.

	Length:	
	Metres.	Feet.
East approach and core-wall	53	174
Solid dam, east	464	1,522
Lock and reclamation	60	197
Sluice-dam	454	1,489
Solid dam, west	662	2,172
Length of masonry dam	1,693	5,554
Length of embankment dam	3,307	10,850
Total length of dam	5,000	16,404

The low-river channel is about 600 metres in width, its greatest depth being about 15 feet. The average discharge ranges from 500 cubic metres

per second in April to about 1,300 cubic metres per second in November, depending on the rainfall in south-west Abyssinia. The sandstone of the *gebel* on the east bank continues across the valley, as shown by the borings put down by the Egyptian Irrigation Department; the finished foundations of the masonry dam and core-wall followed very closely the levels of the rock disclosed by these borings. The cultivated land on the west bank of the river is flat and intersected by two shallow drainage *khors* (Fig. 1, Plate 1), the surface being cotton-soil or a mixture of silt and clay, the subsoil overlying the rock for a considerable depth, being dirty sand and fine gravel. This land is submerged every year in August, September, and part of October, by the Blue Nile flood, and after the latter's subsidence in November, the natives plant a cereal crop. By arrangement with the Egyptian Government, the Sudan Government Railways constructed a branch line from Khartoum to a handing-over station about 1 kilometre east of the *gebel*, so that direct railway communication was established with the Sudan railway system, and more particularly with the granite quarries and the sea at Port Sudan. Passenger traffic across the desert between Khartoum and the dam was conducted by motor-cars and local motor-buses.

Materials for the Works.

Sandstone.—Sandstone was used extensively in the embankment dam, and was obtained from quarries opened out on the east bank for the supply of building stone to Khartoum, and also on the west bank, good stone being procured from a site 8 kilometres west of the dam. The stone exposed in the quarries was variable in quality, light and soft, and absorbed up to 8 per cent. of water, but the stone laid bare in the foundations of the dam, when excavated to secure a sound foundation, was of a much harder nature and better quality, with occasional layers of what was termed mud-stone. The weights and crushing strength of this type of stone are given in Table II.

Table II.

	Specific gravity: lb. per cubic foot.	Crushing strength: tons per square foot.
Stone from quarries used for building in Khartoum	125	212
Fine sandstone	135	250
Mudstone	155	230

It will be evident that this rock was not an ideal foundation for a masonry gravity dam, but in view of the small head of water and the relatively wide section of the dam, the pressures on the rock were inconsiderable.

The maximum pressure on the foundations, with the reservoir full to +380·00 metres and the downstream side dried off, is 2·57 tons per square foot both in the solid dam and in the sluice-dam toes.

Granite.—The whole of the granite required for the works was obtained from Sileitat, about 16 miles north of Khartoum, near to the Sudan Government Railway, a branch-line being built to the quarries and a handing-over station being established. The stone was transported by the Sudan Government Railway, which provided all rolling-stock, and charged 14 *piastres* (2*s.* 10½*d.*) per metric ton for the haulage of about 48 miles. The contractors did the loading-up and unloading. The granite was light grey in colour, and was fine-grained and free from veins, weighing 164 lb. per cubic foot, and the average result of ten crushing tests gave 1,680 tons per square foot. The dressing of the stone was all executed at the quarries by Italian and Sudanese masons. Water was pumped from the Nile, about 7 miles away. The dressing of the granite throughout was excellent. On arrival at the dam the stones were sorted and stacked according to their various types in the granite-yard.

Cement.—All cement required for the works was manufactured in Egypt. It was sent about 170 miles by rail to Suez, thence by steamer about 690 miles to Port Sudan, and thence was sent 515 miles by the Sudan Government Railways to the site. The Gebel Aulia dam is the first major public work of importance that has been constructed throughout with Egyptian portland cement. The cement was manufactured with up-to-date plant in accordance with the British Standard Specification, and every consignment was tested and analysed at the manufacturer's works by a resident representative of Mr. R. H. Harry Stanger, Assoc. M. Inst. C.E. In addition, a sample from every tenth consignment was sent to London and tested in the laboratory; the average results of the forty tests thus carried out are given below :—

Fineness, on 170 × 170 sieve	5·42 per cent.
,, ,, 72 × 72 ,, ...	0·18 ,,
Initial setting-time	151 minutes.
Final ,, ,, ...	229 ,,
Expansion	1·26 millimetre.
7-day tensile test, neat cement	1,238 lb. per square inch.
3-day ,, ,, with sand	476 ,, ,, ,,
7-day ,, ,, ,, ,, ...	577 ,, ,, ,,

Altogether about 100,000 tons of cement were delivered at the site.

Sand.—Sand was obtained from a *khor* on the west bank of the river about 10 kilometres (6 miles) upstream of the dam, and was transported thence by a light decauville railway. Sand was also obtained from a sandbank a few kilometres downstream of the dam, which was transported by local sailing boats. In both cases the sand was of good quality, and only on rare occasions did it require washing.

Concrete Aggregate.—The concrete aggregate was granite, which was crushed and graded at the quarries to the requisite sizes. There was also

a small stone-crushing plant at the dam for occasional use. The results of crushing tests on cubes of 4 : 1 and 6 : 1 concrete, and of tests on the mortar used in building the rubble masonry of the dam, are given in Appendix II.

Solid Dam.

The solid dam (Fig. 3, Plate 1) was constructed of granite rubble masonry in 3 : 1 cement mortar and faced with granite mosaic, the facing-stones being roughly shaped to hexagonal form with their faces hammer-dressed to give a rock-face varying between 2 and 7 centimetres. Header-stones about 75 centimetres long were incorporated at intervals not exceeding 1 metre. The foundations were in all cases taken down not less than 1 metre into the rock, but in many situations where the surface-rock was unsatisfactory, considerably greater depths were excavated. At the upstream heel of the dam there is a cut-off 2 metres deep below the dam foundations, 1·50 metre in width, built with granite rubble masonry in 2 : 1 mortar; for a height of 60 centimetres, the foundations of the dam were also built with granite rubble masonry in 2 : 1 mortar. Before deposition of the foundations, a layer of 2 : 1 mortar was spread over the surface of the rock to ensure a solid bond between the rubble masonry and the rock.

A little trouble was experienced in building the rubble masonry in the cut-off, owing to drainage and seepage-water down the face of the excavation. This was overcome by constructing small dry rubble drains on each side of the cut-off foundations, which were subsequently grouted solid under pressure through steel stand-pipes.

On both faces of the solid dam, three courses of heavy ashlar were provided to form corbels and to provide as wide a roadway as possible; these stones were rock-faced with margin drafts, and presented a pleasing and bold appearance. The roadway was 4·50 metres (14 feet 9 inches) in width between the parapets, sufficient to allow two motor-cars to pass, and was surfaced with granolithic paving falling towards the upstream side, where drainage arrangements were provided for the discharge of surface-water clear of the dam-face. The roadway parapets consisted of three courses of fine rock-faced granite ashlar with margin drafts and a capping of fine-picked, axed, and drafted granite, the latter being carefully dressed and finished with chamfers on all four arrises.

At the east end of the masonry dam a junction was formed with the sandstone of the *gebel* by constructing a core-wall 3 metres in width into the hill, backed on the upstream side with "clay" filling. Two wing-walls with pitched slopes extending eastwards were also built, forming a wide V-shaped approach. A macadam road from the colony to the dam was made and a car-park was provided.

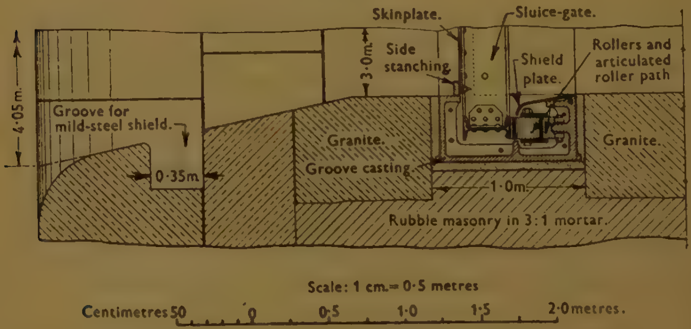
At the western end of the masonry dam, the junction between the

latter and the embankment dam was formed by the construction of two wing-walls to retain the filling and slopes of the embankments. The downstream wing-wall was prolonged about 35 metres beyond the downstream toe of the embankment dam, to divert any rush of storm-water away from the foundations of the masonry dam. A large rubble-masonry buttress was built on the west side of the wing-walls, and both lines of the mild-steel sheet-piling of the embankment dam were incorporated into this to form a cut-off, and a 3-metre by 1·50-metre key with bituminous joggles was introduced between the concrete of the embankment core-wall and the masonry of the dam.

SLUICE DAM.

The construction of the sluice dam (Figs. 4 and 5, Plate 1) was executed on similar lines to the solid dams, except that it was pierced for sixty sluice-vents; the ten western vents, however, were not fitted with gates, being

Fig. 6.



SLUICE-GATE GROOVE.

blocked with heavy reinforced-concrete diaphragms. The width of the roadway between parapets was 5·85 metres, which allowed two motor-cars to pass, after the space occupied by the sluice-gate operating-winches had been deducted. Upstream and downstream of each vent, safety grooves were provided for the steel shields required for drying-off a vent. The walls of the vents were lined throughout with granite ashlar, quoins being provided at all angles, and the floors were paved with dressed-granite paving, at a slope of 1 in 10. The upstream entrance to each vent was tapered from 4·05 metres to 3 metres in width (*Fig. 6*), the latter being the standard width of the openings, which are 4·5 metres in height. The arches of the vents are turned with fair-picked granite-ashlar voussoirs at each end, between which 4 : 1 fine granite concrete was deposited. The upstream faces of the sluice-wells are arched, having horizontal-coursed dressed-granite voussoirs, but the other faces of the wells are ordinary

mosaic granite-facing. Four bastions are built on the downstream face of the sluice-dam, a small one over each of the existing three training-walls, and one large one at the west end of the sluice-dam over a future training-wall, to be built should the ten western vents ever be fitted with sluice-gates and brought into use. A store for tools and gear was constructed on the latter bastion, and another storeroom was built at the west end of the masonry dam. A recess covered by removable chequer-plating was formed in each of the three small bastions for the storage of the safety shields.

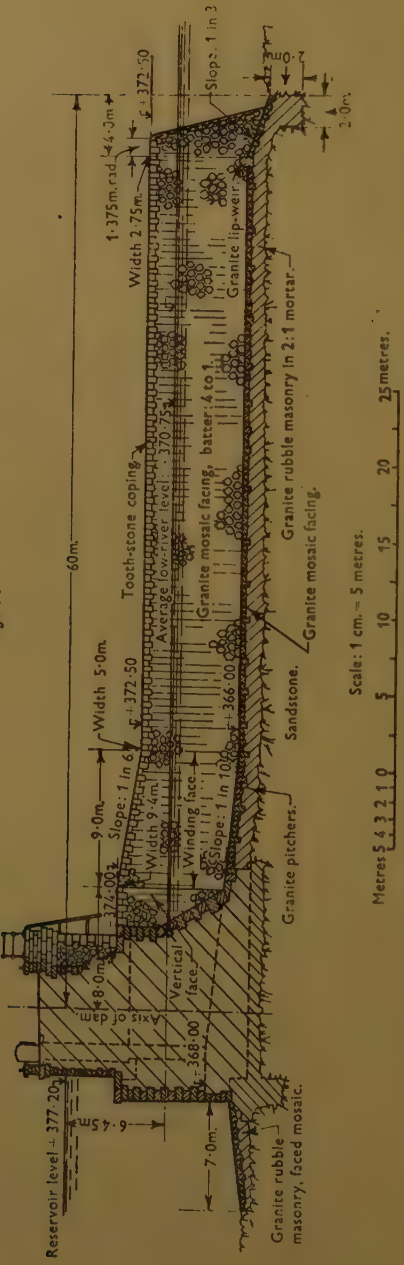
Training-Walls and Aprons.

Three training-walls (Fig. 5, Plate 1, and *Fig. 7*), about 53 metres long, were constructed downstream of the sluice-dam, dividing the fifty sluice-gates into three groups of ten, twenty, and twenty (Fig. 1, Plate 1) respectively, the former group being next to the lock. The training-walls were founded on rock, built in granite rubble masonry and faced with mosaic, the tops of the walls being 1·75 metre above downstream low-river level. Between these training walls, granite-faced aprons were laid, the upper portion being sloped at 1 in 10 to a point 17 metres from the axis of the dam, and surfaced with dressed granite pitchers in continuation of the dressed paving-stones in the vents. Downstream of this pitching, the aprons are of granite rubble masonry, about $1\frac{1}{2}$ metre in thickness, faced with mosaic. At the downstream extremity is a dressed granite anti-scour lip-weir projecting 50 centimetres above the surface of the aprons. At the toes of the aprons the granite rubble masonry was formed to a slope of 1 in 3, being in all cases joined up securely to the existing surface of the sandstone rock. At the west end of the sluice-dam, in continuation of these aprons, a reclamation (Fig. 1, Plate 1) faced with a pitched rubble toe was formed to preserve the regime of the river and to prevent a dangerous eddy forming against the face of the dam. This reclamation was brought up well above low-river level to practically the same level as the training-walls. As the sills of the sluices on the upstream side of the dam are so close to the surface of the rock, a small granite rubble-masonry apron 7 metres wide, and of an average thickness of 75 centimetres, was constructed to minimize the danger of eddies eroding the surface of the rock close to the upstream face of the dam.

Lock Guide-Walls and Fish-Ladder.

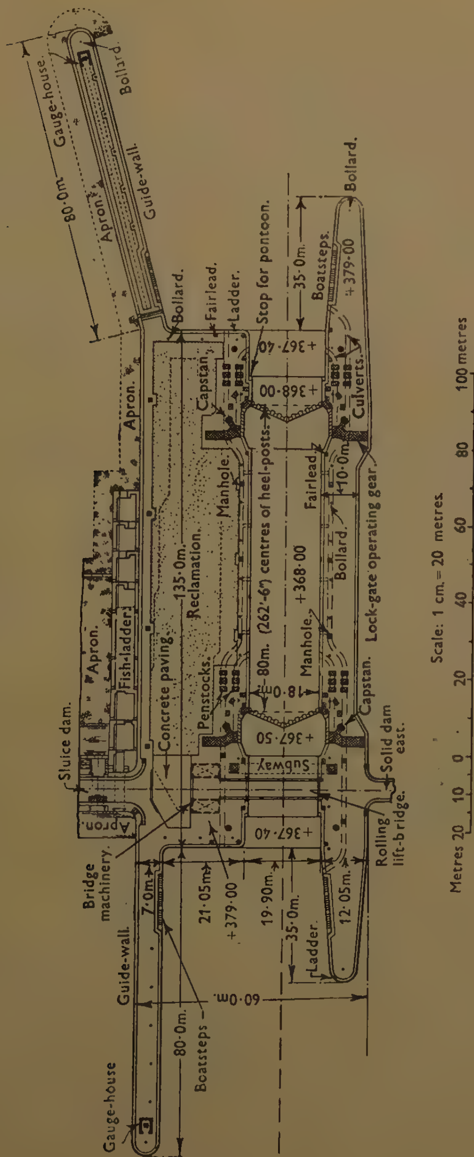
The lock is 80 metres long between heel-posts, 18 metres in width (*Fig. 8*), and 135 metres over the floor. There are two culverts in each wall, one for scouring the backs of the gate-recesses, and the other for equalizing the level of the water by means of four small branch filling culverts in each wall. The floor of the lock is not less than 2 metres in thickness, built in

Fig. 7.



APRON AND TRAINING WALL.

Fig. 8.



Plan of Lock and Fish-Ladder.

anite rubble masonry and vented at intervals, the foundations being . solid rock. A long guide-wall is provided both upstream and down- ream of the lock to form lay-bys for craft and for their protection om the river-current passing through the sluices, more especially for e stern-wheel river-steamers which sometimes push six or seven

barges in front of them; these barges have to be disconnected and await their turn to pass the lock. There is a gauge-well and gauge-house at the extremity of each guide-wall, and automatic self-recording water-level indicators have been installed. The lock and guide-walls have the usual complement of bollards, capstans, fairleads, ring-bolts, ladders, boat-steps, etc. To the west of the wall, a small reclamation has been formed to provide an area on which offices and accommodation have been constructed for the working staff. Immediately to the west of this reclamation is the fish-ladder, which consists of nine water leaps, each 70 centimetres in height; at each leap there is a large pool, and the pools are connected by sloping concave weirs to reduce the white or aerated water to the minimum. The top three weirs are formed by steel Butcher weirs with concave overflows working in cast-iron side grooves, which can be adjusted to suit the level of the impounded water. At the upper end of the ladder the fish have to pass through a tunnel formed in the sluice-dam, the upstream end of which is controlled by a hand-worked steel gate. The depth of water in the pools is about 6 feet, and there is about 2 feet depth of water over the weir at each leap.

Embankment Dam.

The embankment dam (Fig. 9, Plate 1), consists of a core-wall 3 metres thick of 6 : 1 concrete-in-mass with an earth embankment on either side. The core-wall was taken down to the rock between steel interlocking sheet-piling, a key-chase being cut in the surface of the rock. The portion of the core-wall constructed between sheet-piling was 2,440 metres in length, and long piles were necessary to reach the rock. The piles were of the joist-and-clutch type, the joists being 15 inches by 5 inches by 39·5 lb., and the clutches 15·5 lb. per linear foot, and they were driven to an extraordinarily good alignment, being well timbered all the way down, with from three to four settings of walings and struts. The concrete in the trench was deposited through a vertical steel tubular shoot, the bottom end of which was kept embedded in the last-deposited concrete, thus disposing of the risk of segregation of the aggregate. The concrete of the core-wall above ground-level was deposited between "Blaw Knox" steel shutters. In the year 1920, at the extreme western end of the dam, some excavation had been carried out, and the rock had been laid bare; the core-wall was formed directly thereon and prolonged westwards into the rising ground for a sufficient distance for the future heightening, should it be carried out. The "clay" filling was obtained from the surface of the adjacent land from the downstream side only; this was excavated, transported and deposited by graders and tractors, which were very efficient, as the weight of the tractors when depositing the soil consolidated the filling, so that there was very little subsequent settlement. The surface of the ground upstream of the dam was not disturbed, in order to avoid the destruction

of the blanket of "clay." A rubble drain was provided downstream of the core-wall, and small rubble cross-drains were provided at intervals of 150 to 200 metres, discharging outside the toe of the downstream slope. The slope of the embankment upstream was 1 in 2, and downstream was 1 in 3. The surfaces of both slopes of the embankment were pitched with sandstone mosaic pitching 40 centimetres in depth, set dry on a bed of crushed sandstone and sand 15 centimetres in thickness. Toes of hand-packed sandstone rubble, with their surfaces of pitched mosaic, were formed at the toe of each slope, the tops of the pitched slopes at roadway-level being finished off with rock-faced saddle-backed copings. On the upstream slope a wave-breaker of heavy sandstone rubble was deposited as a protection against wave action when the reservoir is full. Three cattle-ramps (Fig. 1, Plate 1) with slopes of 1 in 7, are provided at intervals of 900 metres to allow cattle to be driven over the dam to the upstream side when the reservoir is empty during the summer. The surface of the roadway on top of the dam is, at the present time, rough macadam, but it is the intention of the Government, when all settlement in the filling has ceased, to provide a tar-macadam roadway, as being a better surface and as a precaution against wash-outs due to the heavy local rainstorms which sometimes occur. A heavy pre-cast concrete curb has been provided to form a guide for motor traffic, and a stop against waves slopping over the crest of the dam during strong southerly winds.

Steelwork and Mechanical Equipment.

Sluice-Gates.—The fifty sluice-gates are of the Stoney free-roller type, having a clear opening 3·0 metres wide by 4·5 metres high. They are mounted in cast-iron grooves, grouted into recesses formed in the side-walls of the sluice-vents and wells (Fig. 4, Plate 1 and *Fig. 6*, p. 336). A sill-casting is provided with its surface flush with the floor of the sluice-vent; a lintel-casting, bolted to the groove castings across the top of the sluice-opening, and a roof-casting at the same level as, but upstream of, the lintel-casting, are also provided to form the top of the rectangular sluice-opening. In order to check the strength of the roof-castings, which support a considerable weight of masonry, the first casting was tested in a hydraulic press with a central load, and sustained 300 tons without fracture. All the castings were in Meehanite cast iron. The articulated mild-steel paths, on which the free rollers run, are bolted to the groove castings, as are the mild-steel shield-plates which are fitted to prevent the water passing through the sluice impinging directly on the gate-rollers. The gates are built of steel joists and channels riveted together and to the skin-plating of the gates. Machined rectangular stanching-bars are provided on all four edges of the gates, the two side bars being adjustable. All fitted bolts about the gates and grooves in contact with water are sherardized.

Each gate is operated by means of a pulley carrying two parts of wire-rope wound on the drum of a hand-winch geared so that under maximum-head conditions the gate can easily be raised by four men. The fifty operating winches, which are totally enclosed and fitted with dust-proof doors, are placed near the upstream parapet and supported on steel beams; the latter carry chequer-plates covering the tops of the sluice-wells. Each winch is fitted with an indicator showing the gate opening, but the operating staff are more inclined to use a primitive dipping rod, which is lowered into the sluice-well until it comes into contact with the top of the gate. An automatic brake is fitted to each winch to prevent the load taking charge.

Safety-Shields.—To enable the sluice-vents to be dried out, four safety-shields are provided, which can be dropped into grooves upstream and downstream of the sluice dam. They are built up of sections and plates with "Linatex" reinforced rubber strips around their edges, which press against the dressed granite faces of the safety grooves.

Cranes.—A travelling portal crane handles the safety-shields, and lifts the gates out of the sluice-wells. It runs on rails fixed to the parapets of the dam, and handles and transports loads of 15 tons at 20-foot radius, or lighter loads at 30-foot radius. The hook can be lowered to the sill-level of the gates if required. The crane is driven by a petrol engine of 60 brake-horse-power, fitted with a hydraulic coupling to prevent stalling of the engine and to ensure smooth handling. The speed of hoisting when handling the maximum load is 20 feet per minute, and the travelling speed is 80 feet per minute. The machinery is housed in a sheet-steel cabin, and a small electric lighting set is fitted to illuminate the load and the roadway of the dam at night.

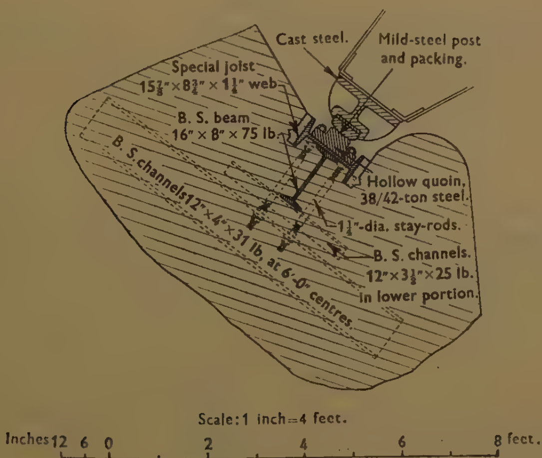
A very useful adjunct to the maintenance establishment at the dam was a petrol electric mobile crane supplied by Messrs. Ransomes and Rapier, Ltd. This was a 3½–5-ton super-mobile crane, capable of lifting 1½ ton at 17 feet radius, 3½ tons at 7 feet, and 5 tons at 5 feet; but for the heaviest load a double snatch-block had to be fitted.

The sluice-gates and gear were supplied and erected by Messrs. Glenfield & Kennedy, Ltd., of Kilmarnock. The travelling portal crane was built by Messrs. Sir William Arrol & Co., Ltd., of Glasgow.

Lock-Gates.—There are two pairs of mitred steel lock-gates having a span of 63 feet between the centres of the heelposts. Each leaf is approximately 34 feet long, 4 feet 8 inches wide, and 34 feet 4 inches high. The gates are double-skinned, and have eleven decks and a buoyancy-space extending up to the eighth deck. The gates have steel sealing-faces on the heel- and mitre-posts, as it has been found that greenheart timber, when exposed to extreme dry heat in the Sudan, splits and warps, and quickly becomes unserviceable. Both the heel- and mitre-posts are fitted with substantial steel castings carrying mild-steel sealing-bars. At the heels these bars engage with high-tensile mild-steel hollow quoins bolted

to an assembly of steel joists and channels concreted into the lock-walls. The quoins and the heelposts are concentric, and to retain the sealing surfaces in correct relation while in use, heavy concave steel castings are provided at the top and bottom of the hollow quoins, engaging with complementary castings at the top and bottom pintles to form hinges over the whole arc of travel. The mitre-posts have two pairs of cast-steel safety-horns, one pair at the top of the gates and the other pair about one-third of the way down, to prevent improper engagement of the mitre-surfaces. They are arranged to come into action if either of the mitre-posts is more than $\frac{1}{2}$ inch out of position. The arrangement of the steel heel-posts, mitre-posts, and safety horns is shown on *Figs. 10* and *11*.

Fig. 10.



Lock-Gate Heel-Post.

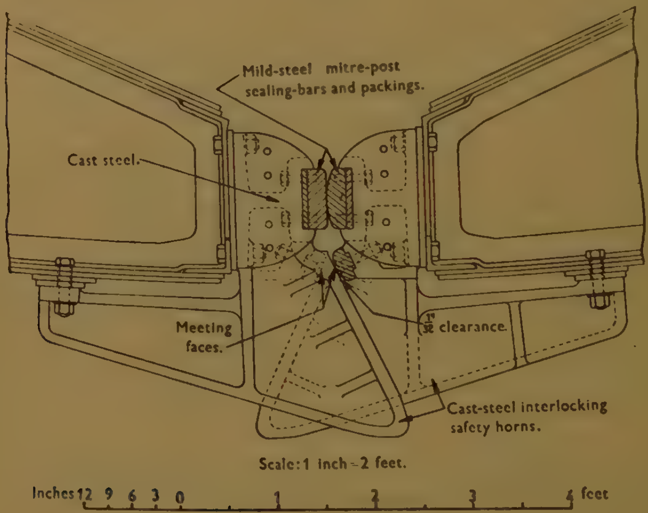
Greenheart timber is fitted on the clapping-sills, as in this position, below water, it is not subject to deterioration. The gates are worked by hand-capstans on the lock-walls, which connect through bevel- and spur-gearing with a rack on the connecting beams of the gates. The operating gears are provided with brakes, but since the gates went into service it has been found necessary to provide substantial wire-rope ties on the downstream side, as at certain periods during strong northerly winds a small reverse head has occurred at Gebel Aulia, when the gates are liable to open slightly and shut rapidly unless they are tied back securely to prevent damage to them.

Penstocks.—There are no sluice openings through the lock-gates, but the culverts referred to in the description of the lock are all equipped with cast-iron bronzed-faced counter-balanced penstocks working on similar bronze faces attached to cast-iron frames. There are altogether eight penstocks, all worked by hand.

23

Pontoons for Lock.—In order to allow repairs to the lock-gates, the lock is provided with two pontoons, which can be placed like caissons against fine-dressed granite stops built in the lock-walls, and so permit the latter to be dried out. A good deal of consideration was given during the design stages of the work to the problem of drying the lock for repairs to the gates, as it is essential that navigation through the lock should be interrupted as seldom, and for as short a time, as possible. Timber dams were ruled out on account of the difficulty of obtaining and keeping suitable timber, the width of the lock, and the time that would be required to construct temporary dams, and it was finally decided to adopt floating

Fig. 11.



LOCK-GATE MITRE-POST.

pontoons. The pontoons are steel structures 61 feet long, 26 feet 3 inches high, and 8 feet 6 inches wide, stiffened internally by decks and diaphragms, and divided into four watertight compartments. The pontoon is towed into the lock in a horizontal position and manœuvred until the rollers with which it is equipped are in contact with the special granite stops, and its axis is at right angles to the lock-walls. Haulage ropes are then connected to the pontoon, led over special fairleads on the lock-walls, and connected to wire-rope blocks operated by the lock capstans, and the pontoon is brought to the vertical position. At very low-water levels this operation must be carried out entirely by the capstans, but at other times a pontoon can be up-ended by adding water ballast to one or two of the watertight compartments. When the pontoon is floating in the vertical position, it is held by guy-ropes and pulleys to the granite-faced stops, and is then sunk by adding water.

When the water-levels in the river require it, portable panels of plating can be added to the top of the pontoon, increasing its effective height by approximately 10 feet. Timber rubbing-pieces are provided to stanch any flow of water around the meeting faces. Two small hand-cranes are provided for handling these portable strakes of plating, and the struts which support them. The pontoons are removed by reversing the operation described, the water being expelled by means of compressed air from two portable petrol-driven air-compressors, which can be connected to the pontoons by armoured canvas hose-pipes. Similar armoured hose-pipes are provided for adding water to the pontoons from the service mains on the lock-walls during sinking operations. A full equipment of haulage and guy-ropes was provided, together with special blocks, fair-leads, and bollards, on the lock-walls. It may be of interest to record that during tests at the site, the operation of placing or removing a pontoon took about 4 hours.

Bridge.—A rolling lift-bridge carries the roadway of the dam across the lock, the bridge being designed to carry a single train of 10-ton lorries, and a pedestrian load of 112 lb. per square foot. The bridge span between bearings is approximately 65 feet, the roadway having a width of 13 feet. It has plate girders, plated quadrants, and an overhead ballast-box supported by the quadrants and bracing members. The bridge is operated by two mild-steel arms carrying rack teeth, engaging with the final pinions of the operating machinery, installed in two steel houses mounted on trestles on each side of the roadway. Power is provided by two 27-brake-horse-power petrol engines, one in each cabin, each having sufficient power to work the bridge at full speed under normal conditions. The engines drive the machinery through hydraulic variable-speed gears, which allow the load to be taken up smoothly, give infinitely variable speeds of working along with increased torques for difficult wind conditions, and prevent stalling of the engines, even if the bridge is driven on to the end-stops. Mechanically-worked control gear is installed in both cabins, all arranged so that the operator can choose either the upstream or the downstream control position, from which he can control the variable-speed gears of either or both engines, and also the locking-bolts at the nose end of the bridge, and the brake. A device is incorporated in the control gear which slows down the bridge to creeping speed when it approaches the limits of its travel, so that it cannot be driven against the stops at full speed; pneumatic buffers are also fitted to the main girders at the nose end. The bridge can be operated in 2 minutes, and there is also a hand-winch which can operate the bridge in an emergency.

The lock-gates, rolling lift-bridge, and the pontoons and other steel-work in connexion with the lock, were supplied by Messrs. Sir William Arrol & Co., Ltd., of Glasgow.

Colony.

A large colony had been built by the Egyptian Government, previous to the commencement of the works, to accommodate the Government supervisory staff, and also barracks for masons, mechanics, etc. The Sudan Government established an excellent hospital, medical staff, post and telegraph office, police-station and police-force. The resident engineer's and contractors' offices were built in an airy position on the shoulder of the *gebel* overlooking the works, and were ready for occupation when the works commenced. The contractors had to build a number of houses, in addition to those handed over by the Government, to accommodate their own staff. A canteen was opened, and a native *souk* or bazaar was built for the supply of goods to labourers and others. Tennis courts and a squash court were also provided.

Execution of Works.

The contractors started work at the site in the autumn of 1933. The extent of the work executed each season is shown in Fig. 2, Plate 1. In December 1933 they commenced structural rubble masonry at the east approach and in the solid dam, east. During the 7 months following, that is, during the first season's work, the whole of the foundations of the solid dam, east, were laid, and the eastern half was practically completed to roadway-level. The foundations of the lock and of the two guide-walls were built, and the walls were completed to about half their height. The foundations under the eastern twenty vents of the sluice dam were also laid. The working seasons lasted from November until the middle of July. During the second season, 1934–35, masonry work was confined to the solid dam, west, and the sluice dam, the walls of the lock and guide-walls being completed as far as was practicable. Work was commenced on the core-wall of the embankment dam in December 1934, and during this season about 600 metres were completed; a few of the piles for this length, however, had been driven during the first season. It had been necessary to provide a navigable channel for vessels passing the works, and during the second season this was situated on the west side of the lock over the site of the first ten sluices, and it was thus maintained till the lock was opened for traffic. During the early part of 1935–36, in the third season, it was possible to complete the sluice dam sufficiently to pass the river through the sluice-openings, thus enabling the western half of the solid dam, east, to be completed; the latter had been left unfinished to allow the river to pass over the foundations already put in. The core-wall of the embankment dam was completed by July 1936. During the last, or fourth, season, 1936–37, which terminated on the 25th April, 1937, when the works were completed, three months before the contract date, there remained

only to finish the parapets and surfacing of the masonry dam, and the completion of the filling, pitching, roadway, etc., of the embankment dam.

Appendix I gives the quantities of the principal items of work handled each season, and their totals in metric units and English equivalents.

The contractors' plant does not call for special remarks, with the exception of the power-station and cableways, which had been used previously on the Nag Hammadi barrage contract[1]. A temporary swing-bridge for rail traffic was erected by the contractors across the navigable channel, and was subsequently taken down and erected across the lock on the line of their temporary viaduct.

The masonry dam in the bed of the river, and for a considerable distance each side of the low-river channel, was constructed inside temporary sudds or dams, the contractors being allowed to use the permanent steel piles of the embankment-dam trench for this purpose, giving a rebate on the price when these were subsequently driven in the core-wall trench. Very few piles suffered damage from temporary use in the sudds.

The contractors obtained direct access to the west bank of the river during a few months by means of a railway across the sudds and the temporary swing-bridge, and over a temporary timber viaduct constructed on the downstream side of the solid dam, east, between the east bank of the river and the lock. In the Author's opinion, it would have been better to have obtained railway communication with the west bank during the first season by means of embankments and a short temporary timber viaduct and swing-bridge on the upstream side of the dam. The contractors would thus have been saved the expense of providing, and working, the two cableways for the transport of granite; moreover, the cableways were inadequate, and had to be supplemented by barge traffic.

Nevertheless, the contractors deserve great credit for having completed the works three months under contract time, and for the excellent workmanship and finish throughout, especially in connexion with the dressed granite.

Operation of Reservoir.

Instructions for operating the reservoir were prepared by Mr. A. D. Deane Butcher, C.B.E., late Director-General Southern Nile, and were published by the Government[2]. As arranged at present, the full reservoir-level of $+377{\cdot}20$ metres will not be reached until 1942, proceeding by stages during the first 6 years. This was arranged with the Sudan government, so that the riparian native population could gradually

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1 "The Nag Hammadi Barrage." Minutes of Proceedings Inst. C.E., vol. 232 (1930–31, part 2), p. 340.

I. W. G. Freeman, "The Aerial Cableways at Nag Hammadi Barrage, Upper Egypt." Selected Engineering Paper No. 120, Inst. C.E.; 1931.

2 "Working Arrangements for Operating the Gebel Aulia Dam." Egyptian Government Press, Cairo; June 1936.

become accustomed to the new conditions, or else to allow time for them to be evacuated to other districts where land would be available.

The length of the reservoir formed by the dam is assumed to be about 350 kilometres, and its width will range from about 4 to 8 kilometres. The average reservoir-area is 1,750 square kilometres, and the average contents are assumed to be 3,250 million cubic metres, measured above the level of the water in the natural bed of the river, as determined by certain gauge readings[1].

The following very briefly describes the program for regulation. The filling of the reservoir will begin every year when the Nile gauge at Atbara on the main Nile records 11·10, usually early in July. What is termed the first filling of the reservoir to level +376·50 metres is then commenced, being attained in normal years by the 20th August. This level is maintained until September, surplus water being discharged through the sluices of the dam.

Should the water-level downstream rise much above the reservoir-level, owing to the natural rise of the Blue Nile, the sluice-gates will be kept open until the level begins to fall, when they will be regulated so as to maintain the reservoir-level at the maximum reached. Should this level be above +377·20 metres, water will be discharged until that level is obtained.

Should, however, the reading on the gauge at Khartoum be below +376·00 metres on the 1st September, a second filling will be commenced by abstracting water from the White Nile, until the reservoir is full to level +377·20 metres, say, early in October; this level will be maintained until emptying commences.

The reservoir is provided solely for replenishing the Aswan reservoir, and for irrigation purposes in Egypt only, the present intention being that the emptying of the Gebel Aulia reservoir will commence on the day after the content of the Aswan reservoir has fallen by 50 million cubic metres of water below its maximum content for that year. This means that the emptying of the Gebel Aulia reservoir will commence about the middle of February. The approximate date when the reservoir should be empty is about the end of April.

Conclusion.

The cost of the works executed under the contract in accordance with the payments made to the contractors was as follows:—

	£E.
Masonry dam and aprons, including sluice-gates, crane, shields, etc.	1,063,600
Lock, guide-walls, fish-ladder, including lock-gates, lift-bridge, pontoons, etc.	361,300
Embankment dam	559,300
	£E1,984,200

[1] See footnote 2, on p. 347.

The cost of the mechanical equipment in connexion with the sluice-gate grooves, sluice-gates, winches, crane, safety shields, lift-bridge, lock-gates, pontoon, capstans, penstocks, etc., included in the above sums, was £E158,420.

A sum of £E750,000 was paid by the Egyptian government to the Sudan government to compensate the riparian native population living on, or in the vicinity of, the site of the reservoir.

The Paper is accompanied by four sheets of drawings, from which Plate 1 and the Figures in the text have been prepared, by eighteen small-scale reproductions of tracings, and by the following Appendixes.

APPENDIX I.

SUMMARY OF QUANTITIES OF WORK EXECUTED EACH SEASON.

	First season 1933–34.	Second season 1934–35.	Third season 1935–36.	Fourth season 1936–37.	Totals.	Totals: English equivalents.
Soft excavation: cubic metres.	483	17,904	51,248	6,385	76,020	99,434 cubic yards.
Rock excavation: cubic metres.	41,990	45,540	7,371	885	95,786	125,288 cubic yards.
Steel sheet-piling: metric tons.	479	3,785	3,805	—	8,069	7,942 tons.
Granite rubble masonry: cubic metres.	83,916	129,702	73,170	—	286,788	375,119 cubic yards.
Concrete-in-mass: cubic metres.	1,880	16,287	86,398	421	104,986	137,322 cubic yards.
Concrete blocks: cubic metres.	—	—	57	639	696	910 cubic yards.
Granolithic paving: square metres.	—	—	7,504	2,151	9,655	11,547 square yards.
Dressed granite: cubic metres.	1,045	6,328	10,962	113	18,448	651,520 cubic feet.
Sandstone coping, pitching, etc.: cubic metres.	—	433	38,970	56,380	95,783	125,284 cubic yards.
Clay filling: cubic metres.	643	59,341	160,791	73,223	293,998	384,550 cubic yards.

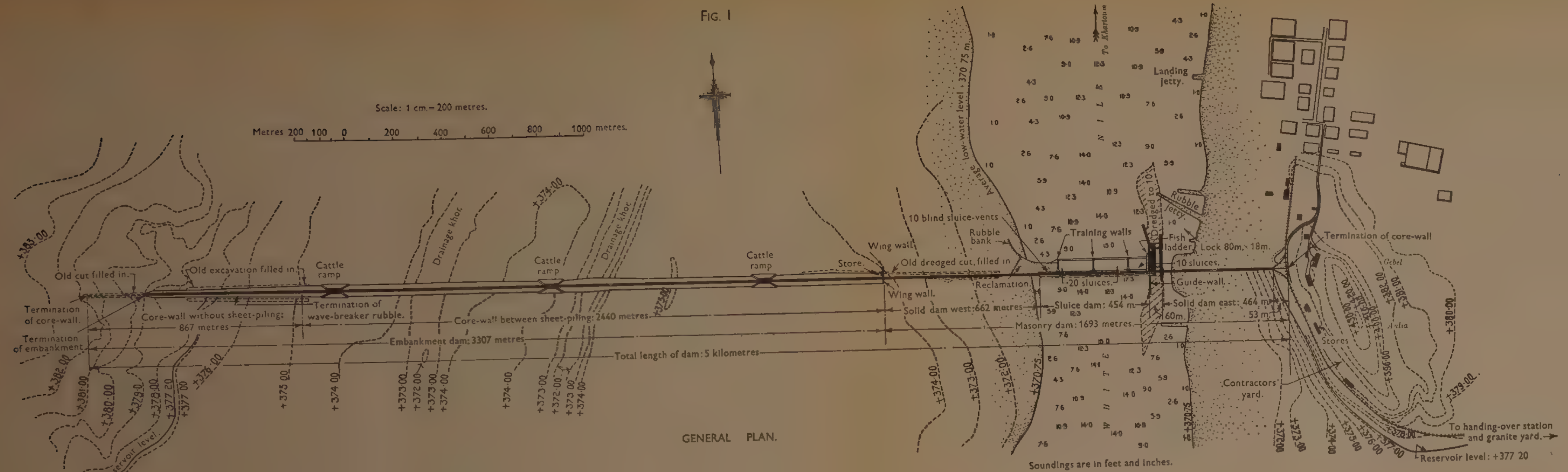
APPENDIX II.

RESULTS OF CRUSHING TESTS ON 6-INCH BY 6-INCH BY 6-INCH CUBES AT 3 MONTHS.

Description.	Average crushing strength: tons per square foot.	Number of cubes tested
4 : 1 fine granite concrete for arches, 1-inch aggregate	371	43
6 : 1 concrete for core-wall, 2 inches granite aggregate	279	25
3 : 1 mortar for building rubble masonry	335	63

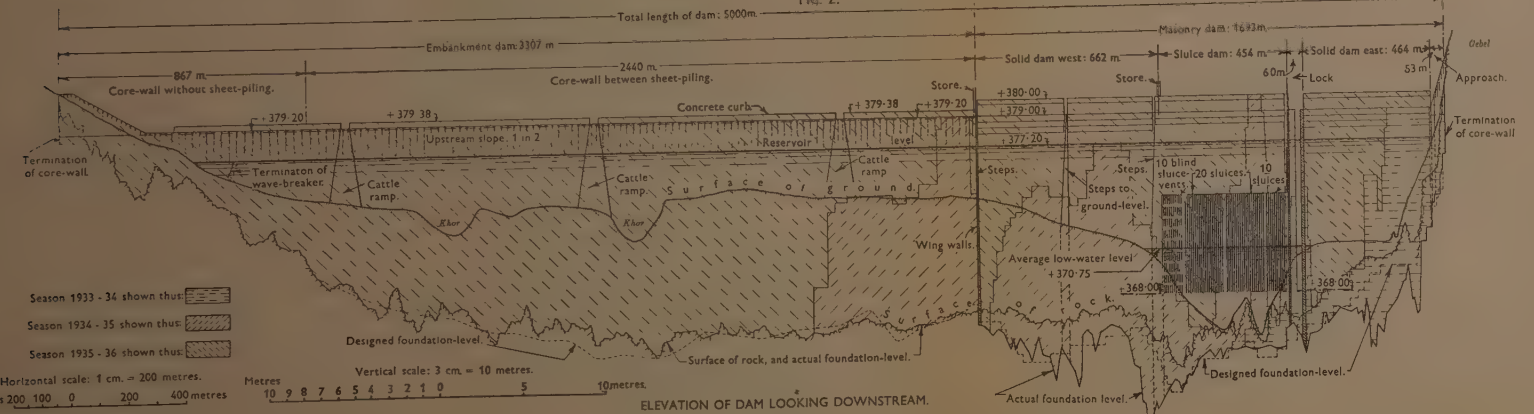
THE GEBEL AULIA DAM.

Fig. 1



GENERAL PLAN.

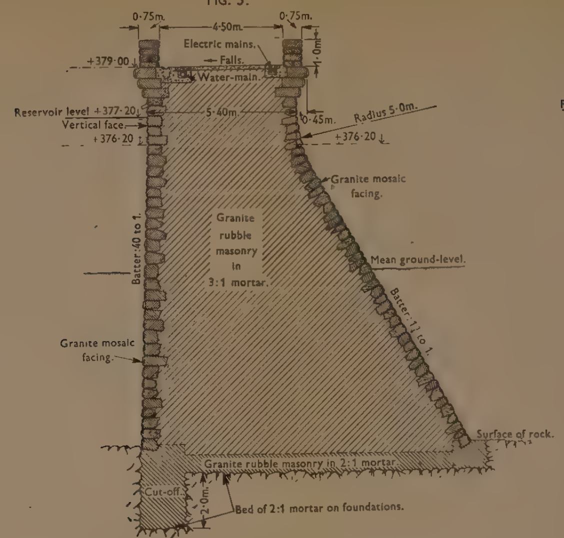
Fig. 2.



ELEVATION OF DAM LOOKING DOWNSTREAM.

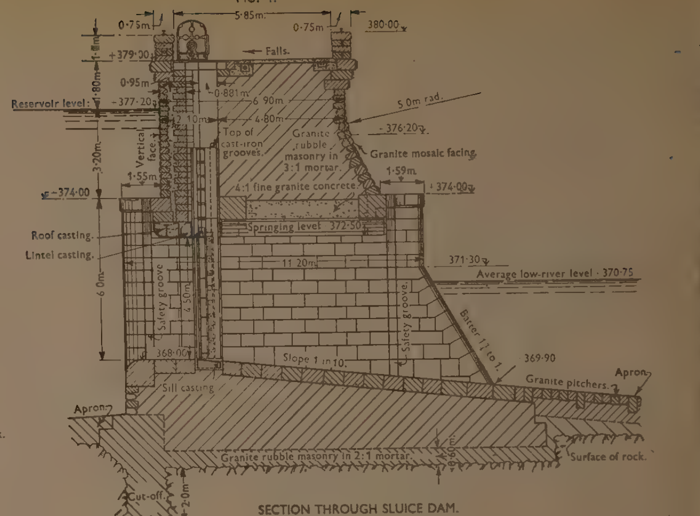
LAYOUT OF DAM

Fig. 3.



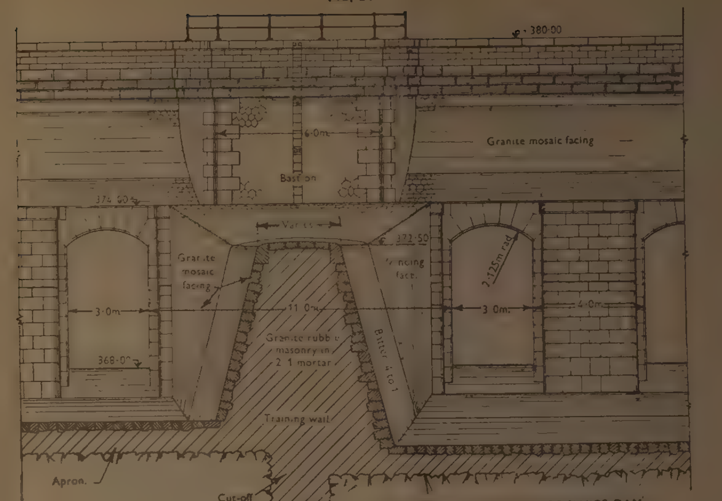
SECTION THROUGH SOLID DAM.

Fig. 4.



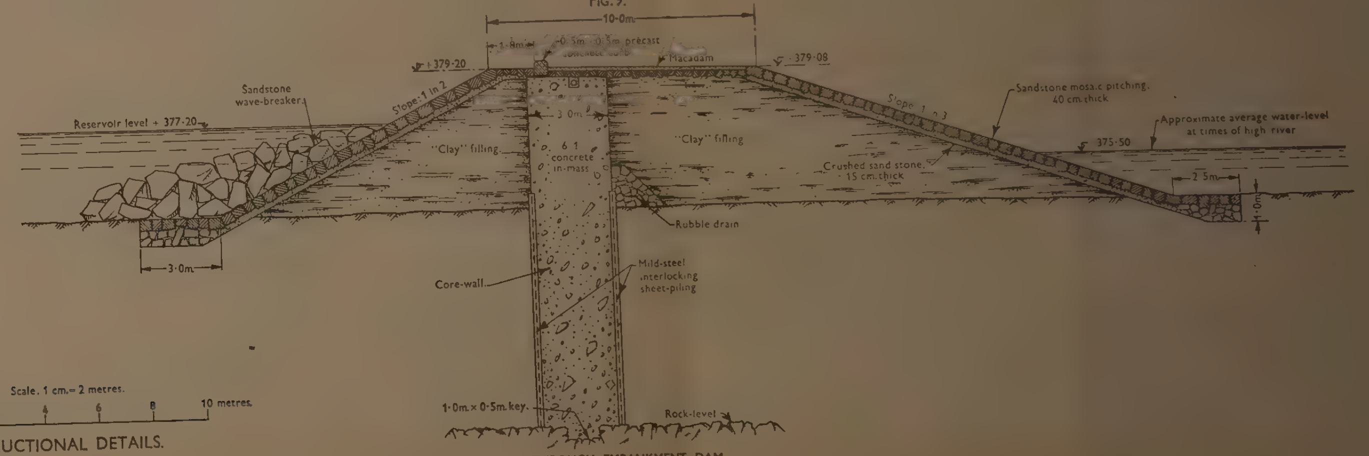
SECTION THROUGH SLUICE DAM.

Fig. 5.



DOWNSTREAM ELEVATION OF SLUICE DAM

Fig. 9.



SECTION THROUGH EMBANKMENT DAM.

Scale. 1 cm. = 2 metres.
Centimetres 200 100 0 2 4 6 8 10 metres.

CONSTRUCTIONAL DETAILS.

A. G. VAUGHAN-LEE.

Paper No. 5248.

"Model Experiments on the Gebel Aulia Dam."

By HASAN ZAKY, Ph.D., B.Sc., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form*[1].)

THE Gebel Aulia dam is one of a series of large storage reservoirs built on the Nile for augmenting its low-season supply in order to cope with the crop demands and allow for a better development of the Nile valley.

Before its actual construction the Egyptian Government wisely decided to submit the proposed plans to model-tests with the object of investigating the whole design thoroughly, eliminating all undesirable features in the structure, and calibrating the sluices for better control.

An extensive series of experiments were carried out for this purpose at the Delta Barrage laboratory. These proved to be highly satisfactory and of great practical value. In many aspects, the final design that was approved and executed was the outcome of these tests.

The models used were seven in number, and the scales ranged from 2 per cent. to full size. The tests can be classified into three categories :—

1. Preliminary experiments for investigating the initial proposals.
2. A general study of the modified and final plans.
3. A precise series for the purpose of calibrating the sluices.

The first series, which could be considered as purely exploratory, were conducted on a $\frac{1}{50}$-scale model comprising two sluices. The study proved beneficial, as it revealed the desirability of some alterations, including an increase in the number of sluices, lowering the sill-level, and fixing the length of the apron.

A second and more extensive series of experiments were then conducted for the purpose of testing the nature of currents, the grouping of bays, and the best shape of training-walls; and for further study of the character of flow near the downstream lock-wall and the fish-ladder. Two $\frac{1}{50}$-scale models comprising more than one bay were used for this study. The tests clearly demonstrated the suitability and effectiveness of the original position of the downstream lock guide-wall, and enabled the consulting engineers to devise two important modifications in their original plan connected with the training-walls and the fish-ladder.

[1] The MS. and the illustrations may be seen in the Institution Library.—SEC. INST. C.E.

The inquiry then proceeded on entirely different lines. Experiments were undertaken for the purpose of precisely calibrating the sluices. This involved the use of four different models. Considering the diversity in the scales used, and the difficulty of proper simulation of the actual position of the upstream and downstream gauges on the different models, the results exhibited close agreement. The experiments aimed at determining accurately the coefficient of discharge for drowned conditions, and establishing clearly the limits for such conditions.

The three small models, namely, $\frac{1}{25}$-scale, $\frac{1}{10}$-scale, and $\frac{1}{3}$-scale, were used for testing gate openings exceeding 1·0 metre for various upstream levels. The largest model (full size) was especially constructed for the study of gate openings of less than 0·50 metre with low upstream levels.

The total number of experiments carried out on these models amounted to about twelve hundred.

The following Table gives the mean coefficients of discharge for drowned conditions: it shows the close agreement of the results obtained from the three models, the coefficient of discharge C being derived from the following equation:—

$$Q = CA\sqrt{H}$$

where Q denotes the discharge, in cubic metres per second per sluice, A the area of the opening, in square metres, and H the difference, in metres, between upstream and downstream levels.

Model scale.	Opening: metres.			
	1·00.	2·00.	3·00.	3·50.
	Coefficient of discharge C.			
$\frac{1}{25}$	3·07	3·14	3·32	3·62
$\frac{1}{10}$	3·02	3·15	3·55	3·81
$\frac{1}{3}$	3·05	3·18	3·50	3·69
Mean coefficient of discharge C	3·04	3·16	3·47	3·70

The tendencies observed earlier in tests made at the Aswan and Sennar dams were confirmed by this investigation; namely, that the coefficient of discharge increases with the gate opening, and that the coefficient is a function of the ratio $\frac{\text{gate opening}}{\text{height of opening}}$. Finally, suitable formulas were derived for the discharge of submerged small gate openings.

The above-mentioned tests served as a useful and reliable base for the correct estimation of the discharge passing through the dam—a matter of vital importance for drawing up the preliminary program of its filling

and emptying. Moreover, the tests helped in settling many points connected with the proper functioning of the reservoir.

In general, the Gebel Aulia dam is the first major structure in Egypt which has been the subject of a detailed technical study through different stages of its construction by the aid of models on various scales, a procedure which proved quite satisfactory, as it resulted in a great saving of time, labour, and money.

The Paper is accompanied by eighteen sheets of drawings and seven photographs.

Paper No. 5254.

"Characteristics of Fire Jets."

By JAMES STANLEY BLAIR, B.Sc.

(Ordered by the Council to be published with written discussion[1]*.)*

TABLE OF CONTENTS.

	PAGE
Introduction	354
Nozzle efficiency	355
Frictional losses in standard fire hose	358
Heights of jets projected at various angles	360
Throws of jets projected at various angles	368
Utilization of the foregoing data	375
Conclusions	378
Appendix	379

INTRODUCTION.

ALTHOUGH jets of water have been used for fire-fighting and other purposes for a very long time, it is surprising how few actual tests have been carried out to determine the characteristics of fire jets. The scanty information which has been published is usually based upon tests carried out in 1888 by Mr. John R. Freeman[2]. In view of the fact that more than 50 years have elapsed since those tests were carried out, it was decided to make a further series of tests dealing with the heights and throws of jets at all angles, and, in particular, to determine the throws obtainable with limited rise of the jet trajectory above the nozzle, such as would occur in fire-fighting in constricted passages or in mine roadways. At the same time, it was considered advisable to make check-tests on the efficiency of commercial fire-jet nozzles, and also on the loss of pressure in lengths of hose. It was decided to carry out the tests with the following standard sizes of nozzle: ½-inch, ⅝-inch, ¾-inch, and 1-inch bore; and the values for the ⅞-inch size were added later by interpolation.

A program of tests was, therefore, arranged to determine:—

(1) Nozzle efficiency and relationship between discharge and pressure.

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[1] Correspondence on this Paper can be accepted until the 15th August, 1941, and will be published in the Institution Journal for October 1941.—SEC. INST. C.E.

[2] "Experiments Relating to Hydraulics of Fire Streams", Trans. Am. Soc. C.E., vol. xxi (1889) p. 303 (Nov. 1889).

(2) Loss of pressure in fire hose.
(3) Relationship between height of jet, angle of projection, and pressure.
(4) Relationship between throw of jet, angle of projection, and pressure.

Nozzle Efficiency.

The method of determining the nozzle efficiency consisted in measuring the quantity discharged from the nozzles at various pressures by means of a calibrated tank.

Type of Nozzle.

The nozzles used were standard fire nozzles of the plain conical type, in which the bore tapered uniformly for a length of about 2 inches from $1\frac{1}{2}$ inch to the outlet diameter, the outlet being then parallel for a further $\frac{3}{4}$ inch. The nozzle was screwed on to the usual conical "branch-piece", which tapered from $2\frac{1}{2}$-inch to $1\frac{1}{2}$-inch bore in a length of about 1 foot. The branch-piece was attached to a length of standard $2\frac{1}{2}$-inch unlined canvas fire-hose. Two pressure-gauges were inserted, one at the junction between the hose and the branch-piece (that is, at the $2\frac{1}{2}$-inch bore) and the other at the junction between the branch-piece and the nozzle (that is, at the $1\frac{1}{2}$-inch bore); and when allowance was made for the difference in velocity-head at these two pressure-points, the values of total pressure obtained from the two gauges were substantially the same.

Shape of Jet.

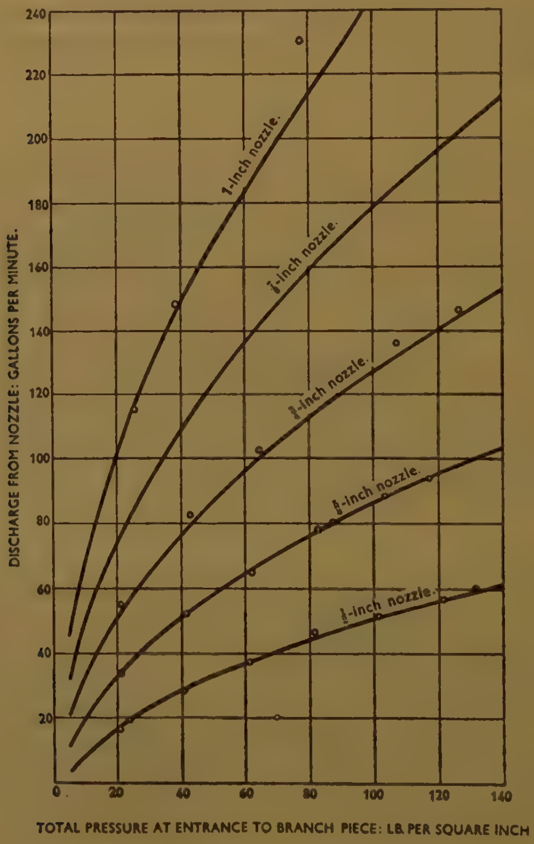
The bores of the various nozzles were measured directly, and also the diameters of the jets emerging from the nozzles at various pressures. These measurements indicated that the jet assumed a plain conical form of very small included angle, and that there was no *vena contracta*. There was no appreciable change in the shape or in the dimensions of the jet near the nozzle for different pressures within the range considered (namely, 10 lb. to 140 lb. per square inch). The minimum diameter of the jet is, therefore, equal to that of the clear bore through the nozzle, and this has been assumed in all calculations.

Test Results.

Fig. 1 illustrates the results of these tests, plotted as the discharge, in gallons per minute, against the total pressure at the entrance to the branch-piece. This total pressure takes into account the velocity-head of the water moving through the hose, although in most cases this is negligible, being seldom greater than 1 lb. per square inch. The curves shown on this graph are obtained from the equation given below, utilizing the values of

efficiency found for these jets. It will be seen that the plotted test results are very close to the values predicted from theory. The efficiency of the various nozzles tested was calculated from knowledge of the size of the clear bore of the nozzle, the volume discharged in a given time, and hence the velocity at exit from the nozzle. This was then compared with the theoretical velocity obtainable from the pressure at the entrance to the

Fig. 1.

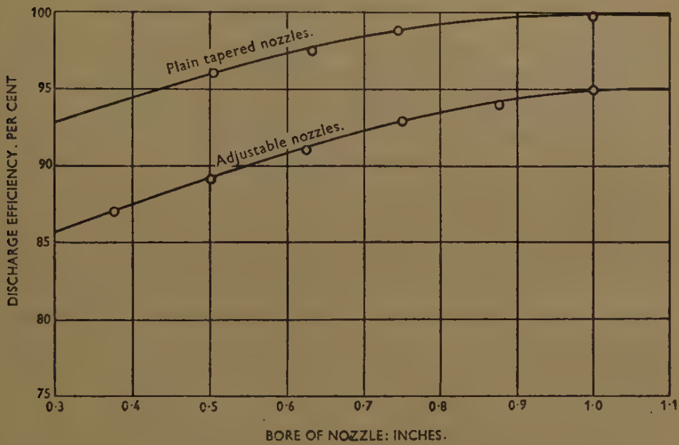


branch-piece. The efficiency considered is, therefore, the discharge efficiency, that is, the ratio between the quantity actually discharged and the quantity which would be theoretically discharged through the nozzle by the conversion of the total pressure into velocity at the exit. The discharge efficiencies, which are shown on *Fig. 2*, were found to be surprisingly high, the larger nozzles being more efficient than the smaller ones. Whilst, of course, it is not possible for a nozzle to be 100 per cent. efficient, the 1-inch

nozzles very nearly attained that efficiency, and even the ½-inch nozzle had an efficiency of 96 per cent.

These results are similar to those obtained by Freeman, who found that the discharge efficiency of jets between ¾ inch and 1 inch in diameter was about 98 per cent. It is possible that the slight increase in efficiency

Fig. 2.



found to-day is due to improved technique in the manufacture of the nozzles.

Adjustable Nozzles.

Values are also given for an adjustable nozzle, tested at the same time: this is of the type in which a spear is introduced into the nozzle-opening, in such a way that the jet forms in the air clear of the nozzle. It is not to be expected that this type of jet will show quite such a high efficiency; moreover, it is difficult to measure the cross-section of the jet exactly, since it is seldom truly circular. However, the principal object of such an adjustable nozzle is to enable the requisite throw or height to be obtained in special situations; consequently, it is not of great importance whether the quantity discharged is exactly equal to that of a fixed nozzle of the same nominal bore.

Formula for Discharge from Nozzles.

There is no evidence that the efficiency of the nozzle is appreciably affected by either the pressure or the velocity of the jet, and the following formula may be used to calculate the discharge, G, in gallons per minute, from any size of jet:—

$$G=24{\cdot}7E_d\, d^2\sqrt{P}\,;$$

where E_d denotes the discharge efficiency, d the diameter of the jet, in inches, and P the total pressure at the entrance to the branch-piece, in lb. per square inch.

This formula is sufficiently accurate to form a convenient means of measuring the discharge of hydrants, fire pumps, etc.

Frictional Losses in Standard Fire Hose.

In practice, exact values of the pressure-loss in fire hose are not of great importance, since friction varies to some extent each time the hose is used, owing to such changing factors as layout of hose, age, difference in diameter between one hose and another, etc. A number of tests were carried out, therefore, with standard 2½-inch hose (actually 2·6 inches bore) and with various layouts. The results of these tests, together with the hose layouts, are shown in *Figs. 3*, upon which is also drawn Freeman's curve for friction in fire hose and a suggested relationship which is more in conformity with formulas for the flow of water in pipes. Although no tests have been carried out on hoses of diameters other than 2½ inches, nor on hoses with different linings, Freeman's tests may be used as a guide, and the following values may be taken as sufficiently accurate for practical use :—

For unlined canvas hose of 2½-inch nominal bore (actual bore 2·6 inches):—

Pressure-drop (lb. per square inch) per 100 feet,

$$p=\frac{G^{1\cdot 85}}{4\cdot 33d^5}=\frac{G^{1\cdot 85}}{520},$$

where G is expressed in gallons per minute and d in inches.

For thin-rubber-lined hose of 2½-inch nominal bore :—

Pressure-drop, 60 per cent. of that for unlined canvas hose.

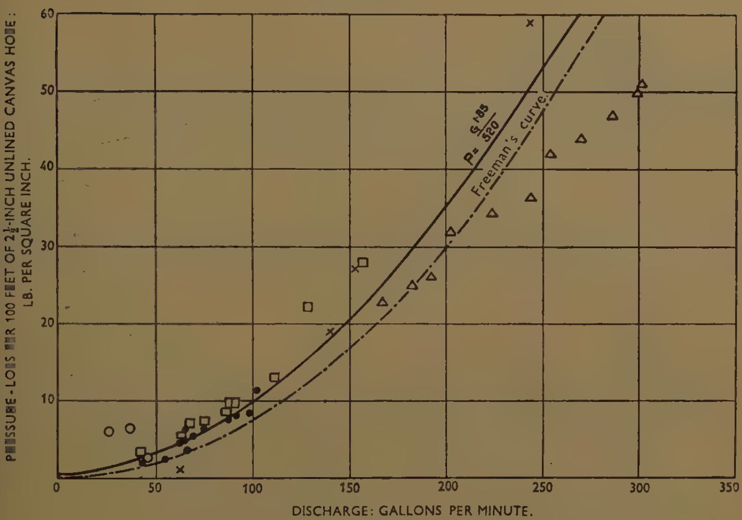
For thick-rubber-lined hose of 2½-inch nominal bore :—

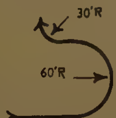
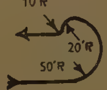
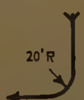
Pressure-drop, 45 per cent. of that for unlined canvas hose.

Between the limits of 2 inches and 3 inches diameter the pressure-loss can be taken as inversely proportional to the fifth power of the diameter.

These formulas include the effect of sinuosity and also that of bends caused by laying the hose around obstacles ; the allowance for each of these effects has been taken as about 5 per cent. of the head lost. Although the rubber-lined hoses show such marked superiority over the unlined hose as regards frictional loss, other considerations, such as weight, cost, ease of handling, etc., render the unlined hose preferable.

Figs. 3.



HOSE LAYOUT	NO. OF LENGTHS OF HOSE.	TOTAL LENGTH OF HOSE: FEET.	PLOTTED AS :–
 6'R	1	68	×
 6'R	1	75	○
 30'R 60'R	4	230	□
 10'R 20'R 50'R	4	230	●
 20'R	2	96	△

Heights of Jets Projected at Various Angles.

Theoretical Considerations.

If a jet of water be projected vertically upwards in a vacuum it will, theoretically, attain a height equal to the head of water acting upon the jet (provided the nozzle is 100 per cent. efficient); that is,

$$h = 2{\cdot}3P,$$

where h denotes the height, in feet, and P the pressure, in lb. per square inch. Similarly, if the velocity be V feet per second, the theoretical height is given by :—

$$h = \frac{V^2}{2g}.$$

In practice, when the jet is projected in air, the friction between the air and the jet reduces the height to which it rises to some fraction of the theoretical height. At the same time, the fact that, generally speaking, the nozzles are not 100 per cent. efficient further reduces the height. In a vacuum, the relationship between the height of the jet and the angle of projection is determined by simple geometrical considerations, since the trajectory of the jet is a parabola. The height at any angle α in a vacuum is given by :—

$$h_\alpha = \frac{V^2 \sin^2\alpha}{2g} = 2{\cdot}3P \sin^2\alpha.$$

The tests described below indicated that in air the actual height at any angle is approximately a fixed proportion of the theoretical height at the same angle, for any one nozzle and pressure.

Method of Testing.

Tests to determine the relationship between theoretical and actual height were carried out as follows :—

The branch-pipe and nozzle were fixed to an adjustable stand so that they could be tilted to any angle, the angle being measured by a protractor-level placed against the branch-piece. The heights of the jet were measured in two ways :—

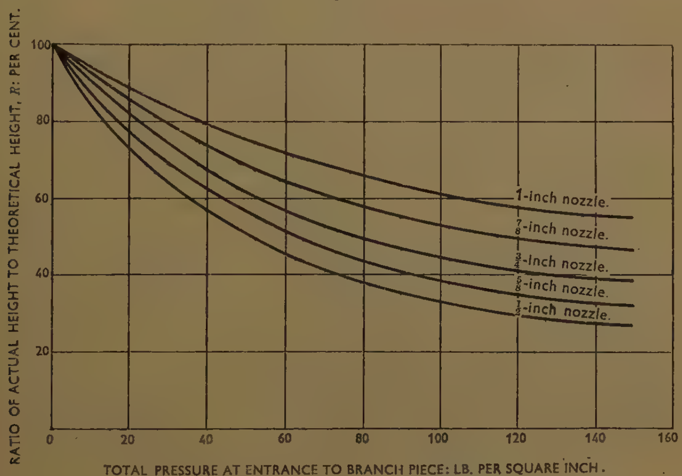
(1) When the height was low, say less than 20 feet, a calibrated pole was used.

(2) For greater heights a meteorological hydrogen balloon, 4 feet in diameter, was utilized to carry a calibrated string about 150 feet long, on which "ping pong" balls were threaded at intervals of 10 feet, whilst the uppermost interval was sub-divided into feet in the same way: these balls formed very

convenient markers, being white and reflecting the light in all directions, no matter how the string might twist. The height of the jet was either measured against the string itself, if it was approximately vertical, or, when a slight side wind caused the string to lie at an angle, estimates were made by holding a marked stick at arm's length and comparing the length along the string with the height of the jet—the observer, of course, standing a considerable distance away.

These methods were considered to be of ample accuracy in view of the fact that the height is, in any case, difficult to determine, owing to the diffused nature of the jet at the top of its trajectory. A question arises,

Fig. 4.



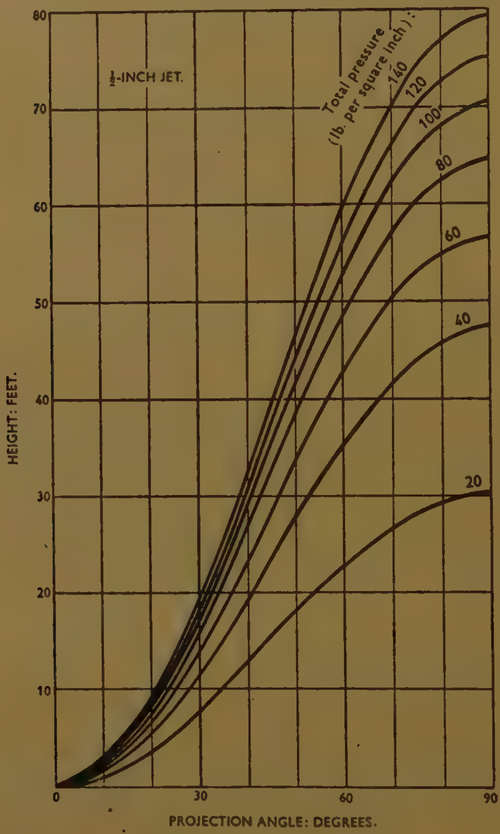
however, as to the exact part of the jet to which the height is to be measured. At the top of its trajectory the jet has usually become rather broken, and, occasionally, some extreme drops may rise considerably above the main portion. Usually, however, and particularly at low angles of projection, the jet remains more or less compact until it attains nearly its highest point. The point to which measurements were made in the tests was approximately half-way between the end of this compact jet and the highest point attained by the drops. Whilst the jet is rather diffused at that point, it still forms a very effective stream for fighting a fire.

A very large number of tests were carried out with the various nozzles at pressures in steps of about 20 lb. per square inch up to 140 lb. per square inch, and at angles of at least every 10 degrees up to 90 degrees. The results were reasonably consistent; *Fig. 4* shows a series of smooth curves obtained from these tests, giving the ratio, R, between the actual and the theoretical heights, and it will be seen that the proportion of the

theoretical height actually attained is progressively less as the pressure increases, and is also less as the jet-diameter decreases.

From these curves have been derived *Figs. 5–9*, which illustrate the

Fig. 5.



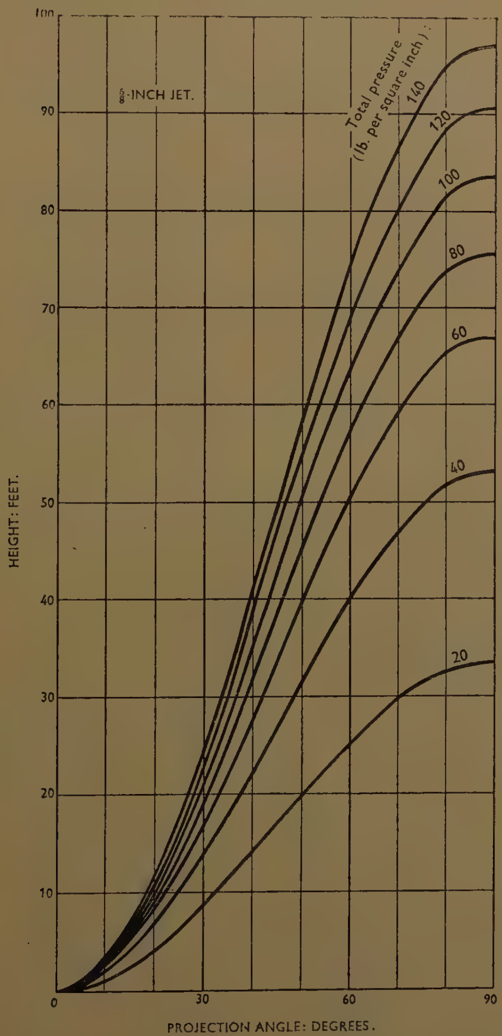
relationship between height, angle, and pressure for each size of jet; these graphs were plotted from the formula :—

$$h_a = 2{\cdot}3RP\sin^2\alpha,$$

where R denotes the ratio of actual to theoretical height shown on *Fig. 4*. *Fig. 10* shows the maximum heights to which the jets can rise, when projected vertically. All of these tests were carried out in a slight cross-wind, of about 5–8 miles per hour and, therefore, probably do not represent the maximum height which could be attained in still air; but still-air conditions are seldom experienced. If stronger winds are encountered, the heights attained will not be so great as those indicated on the graphs. In

any case, the values shown are subject to considerable variations, owing not only to the factor of wind, but also to the individual decision as to the correct point of the jet to be considered as the end of the effective jet.

Fig. 6.



The principal advantage of these graphs is that, whilst they are, perhaps, not exactly accurate as regards the actual height attained, at least they show the relative heights attained for different pressures or for different angles.

Fig. 7.

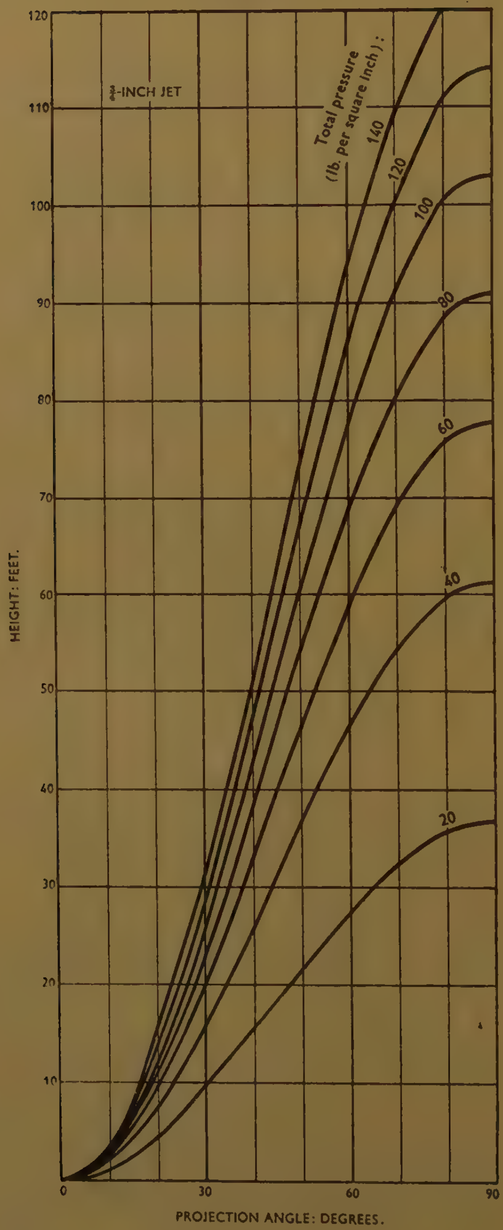
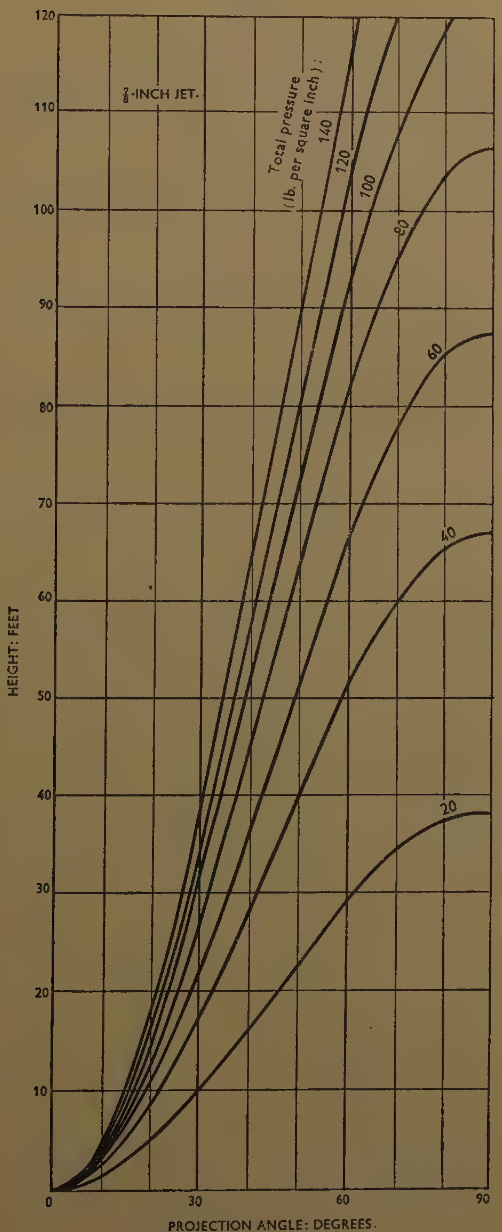

¾-INCH JET
Total pressure
(lb. per square inch):
140
120
100
80
60
40
20
HEIGHT: FEET.
120
110
100
90
80
70
60
50
40
30
20
10
0
30
60
90
PROJECTION ANGLE: DEGREES.

Fig. 8.



7/8-INCH JET.
Total pressure (lb. per square inch):
140
120
100
80
60
40
20
HEIGHT: FEET
PROJECTION ANGLE: DEGREES.
0
30
60
90
10
20
30
40
50
60
70
80
90
100
110
120

Fig. 9.

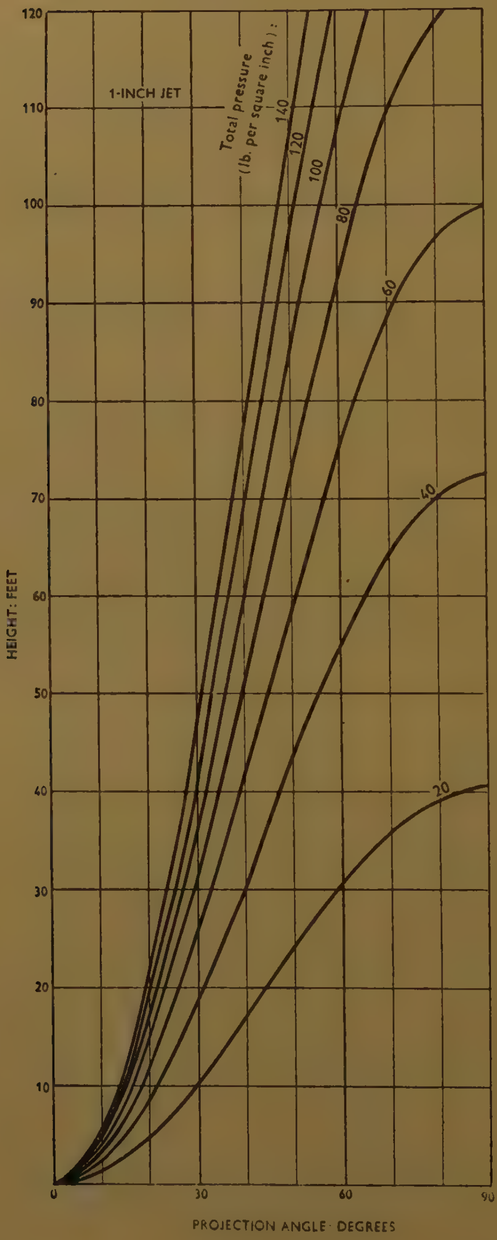
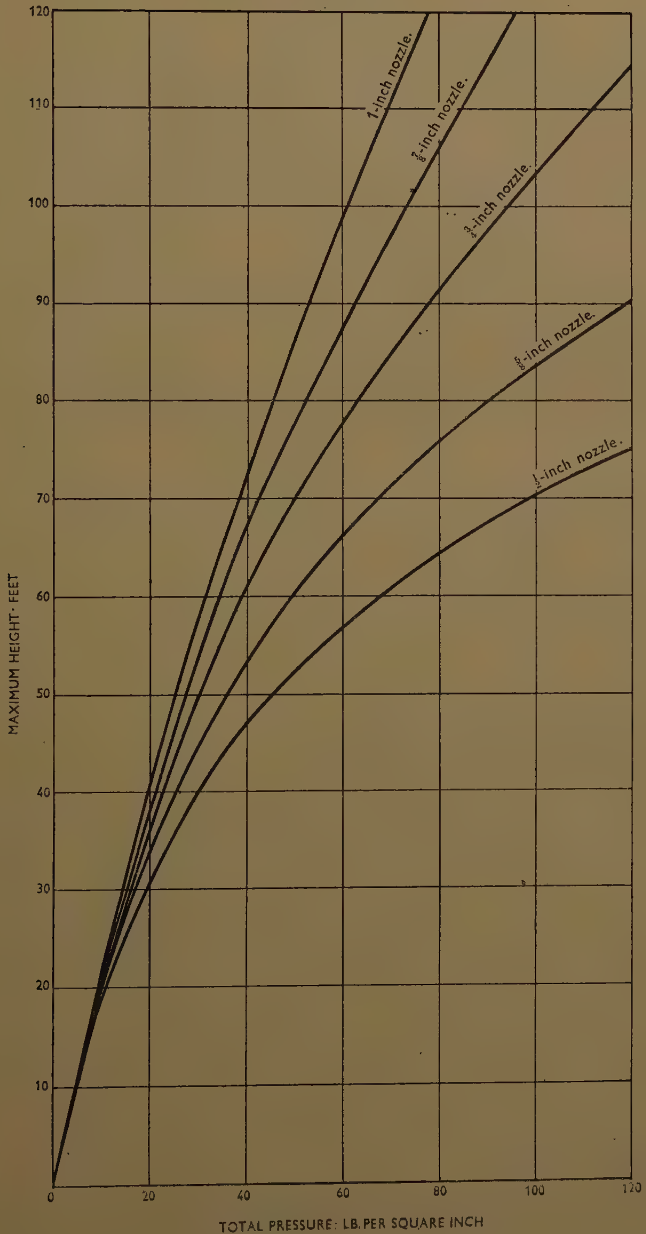

1-INCH JET
Total pressure
(lb. per square inch):
140
120
100
80
60
40
20
HEIGHT: FEET
120
110
100
90
80
70
60
50
40
30
20
10
0
30
60
90
PROJECTION ANGLE· DEGREES

Fig. 10.


1-inch nozzle.
7/8-inch nozzle.
3/4-inch nozzle.
5/8-inch nozzle.
1/2-inch nozzle.
MAXIMUM HEIGHT · FEET
TOTAL PRESSURE: LB. PER SQUARE INCH
0
10
20
30
40
50
60
70
80
90
100
110
120

It will be seen from these graphs that there is no evidence to support the rather common idea that with a given nozzle, increasing the pressure beyond a certain point will cause no further increase in height of the jet, and will, in fact, definitely cause a decrease. This misconception seems to have arisen from extrapolation of published formulas beyond the test data to which they apply. It is true that the increase in height for a given increment of pressure becomes less and less, but there is still an appreciable increase in height for increase in pressure, at least up to 140 lb. per square inch with the jets tested.

Values for the heights of jets at pressures of up to 100 lb. per square inch have been published from time to time; but apparently this is the first occasion on which actual tests on vertical jets with pressures exceeding about 60 lb. per square inch have been described. Freeman's original tests on vertical jets did not extend beyond pressures of 60 lb. per square inch, and the higher values given by him were extrapolated from data on inclined jets.

Throws of Jets Projected at Various Angles.

Theoretical Considerations.

In a vacuum, the form of the jet would be a parabola, and the throw T would be given by :—

$$T = \frac{4h}{\tan \alpha} = 4{\cdot}6 P \sin 2\alpha$$

where h denotes the maximum height of the jet at a given projection-angle α, and P denotes the pressure, in lb. per square inch.

Under these conditions the maximum throw will, of course, occur at an angle of 45 degrees. In practice, in air, the shape of the trajectory is not parabolic, the jet being shortened considerably owing to the fact that the water tends to break up and fall abruptly after about half the trajectory has been traversed. As a result of this change of shape, the angle for the maximum throw is reduced to considerably less than 45 degrees, being about 25 degrees for pressures of up to 60 lb. per square inch and diminishing to only 12 degrees for pressures of the order of 140 lb. per square inch.

The effect of wind upon the throw of jets is very appreciable. As mentioned on p. 362, *ante*, the tests were carried out in a cross-wind of about 5–8 miles per hour; consequently, in still air rather higher throws might be expected.

Method of Testing.

The tests on the relationship between throw and angle of projection were carried out at the same time as, and with the apparatus used for, the height tests. The throws were measured from the nozzle to the point at

Fig. 11.

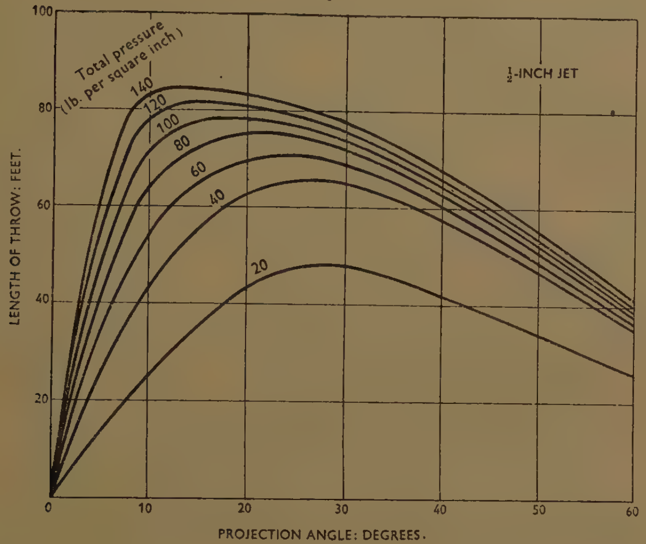
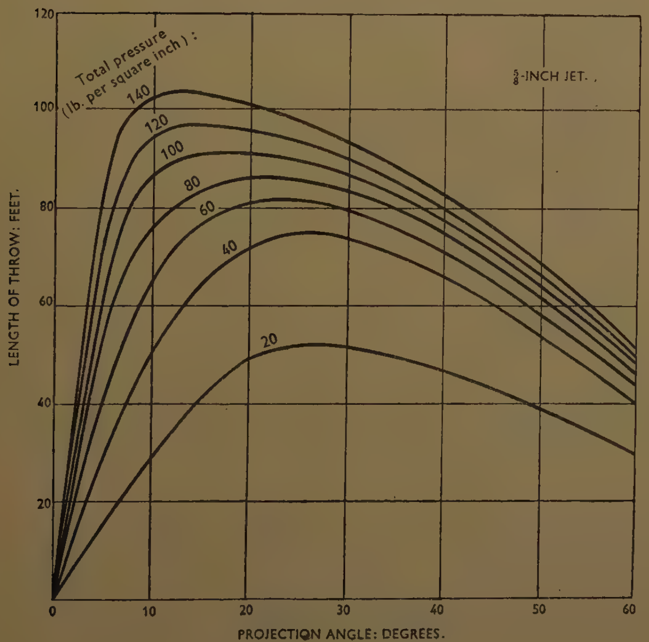
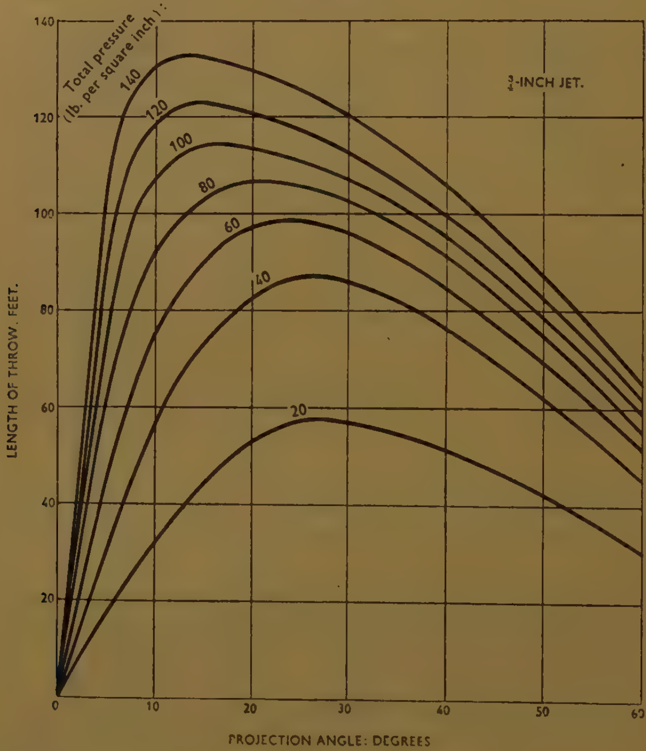

Total pressure (lb. per square inch)
140
120
100
80
60
40
20
½-INCH JET
LENGTH OF THROW: FEET.
100
80
60
40
20
0
10
20
30
40
50
60
PROJECTION ANGLE: DEGREES.

Fig. 12.


Total pressure (lb. per square inch):
140
120
100
80
60
40
20
⅝-INCH JET.
LENGTH OF THROW: FEET.
120
100
80
60
40
20
0
10
20
30
40
50
60
PROJECTION ANGLE: DEGREES.

which the jets cut the horizontal plane passing through the nozzle. Here again, the exact length of throw was rather difficult to determine, since the jet did not fall on one clearly-defined spot, but, particularly at the higher angles of projection, was diffused over a considerable area. The approximate centre of this area was taken in all cases, and it is considered that the jet at this point is reasonably effective for fire fighting. As in the case of the height tests, occasionally extreme drops of water will be thrown

Fig. 13.

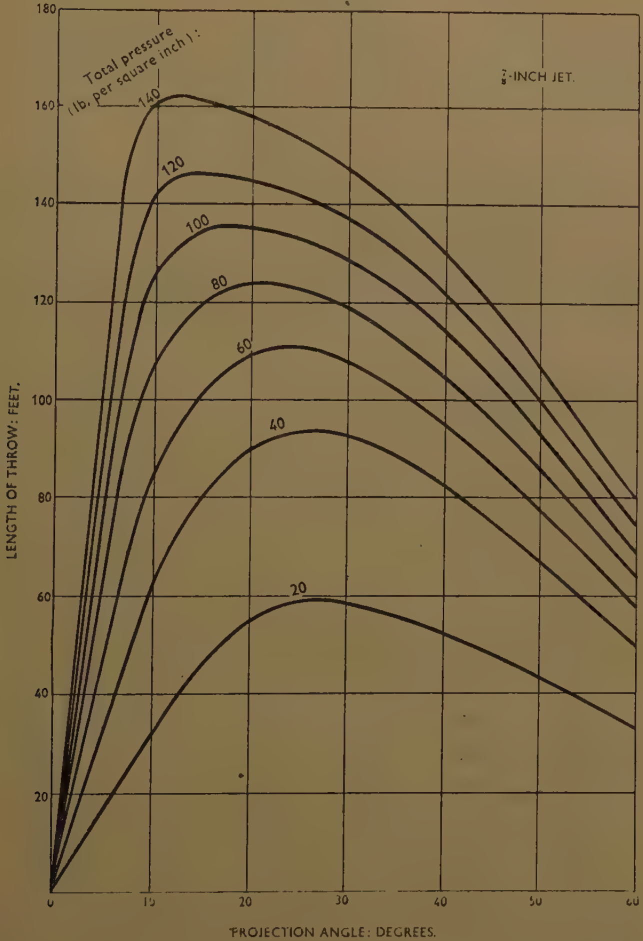


to appreciably greater distances than the values given. *Figs. 11–15* indicate the values of the throw for different angles of projection for the different nozzles tested, and *Fig. 16* indicates the maximum throws attainable.

As previously mentioned, it will be seen that the throw at first increases almost linearly with increase of angle, but after an angle of about 10 degrees has been attained the throw does not increase so rapidly and a maximum throw is attained at an angle of about 15–25 degrees, instead of at an angle of 45 degrees, as would occur in a vacuum. After the maximum has been attained, the throw declines more or less steadily with increase of angle;

but the graphs have not been continued beyond an angle of 60 degrees, since for higher angles of projection the throw is not very important, an equal throw being obtainable for a smaller projection-angle, whilst the jet with the smaller angle is much more compact and effective. Also, of

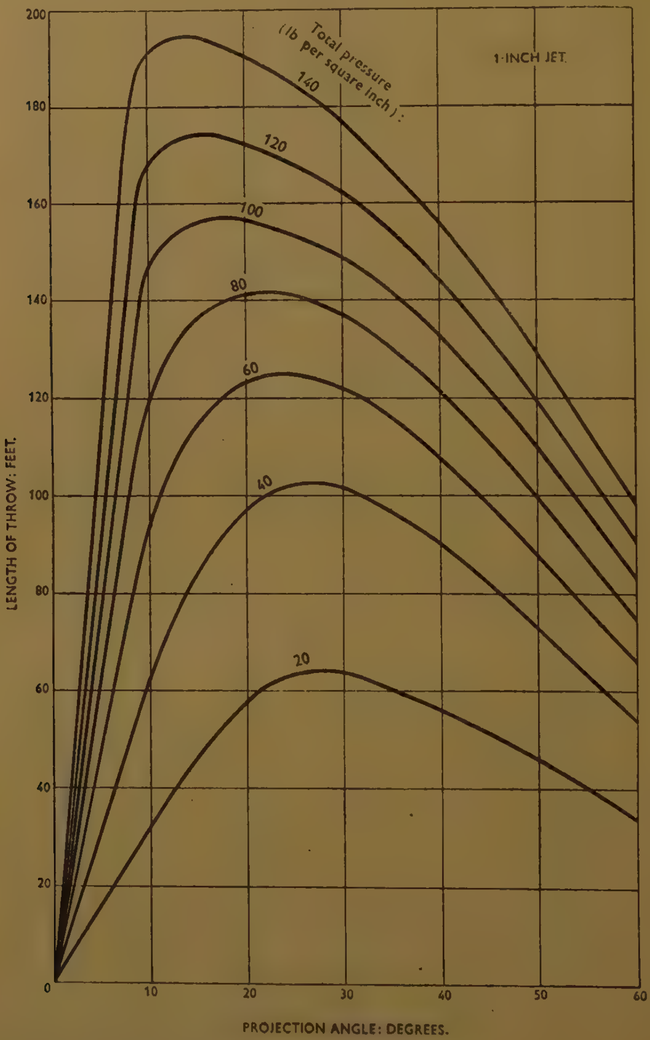
Fig. 14.



course, for higher angles of projection the effect of any wind upon the throw is much more important. It must be remembered that the actual values obtained from these tests vary very considerably among themselves, and the graphs shown represent families of smooth curves derived from the tests. It is only by taking a very large number of test results and averaging them, that such smooth families of curves can be obtained, and

it cannot be expected that actual test results will agree exactly with the values given, since so many factors—in particular, atmospheric conditions—influence the results considerably.

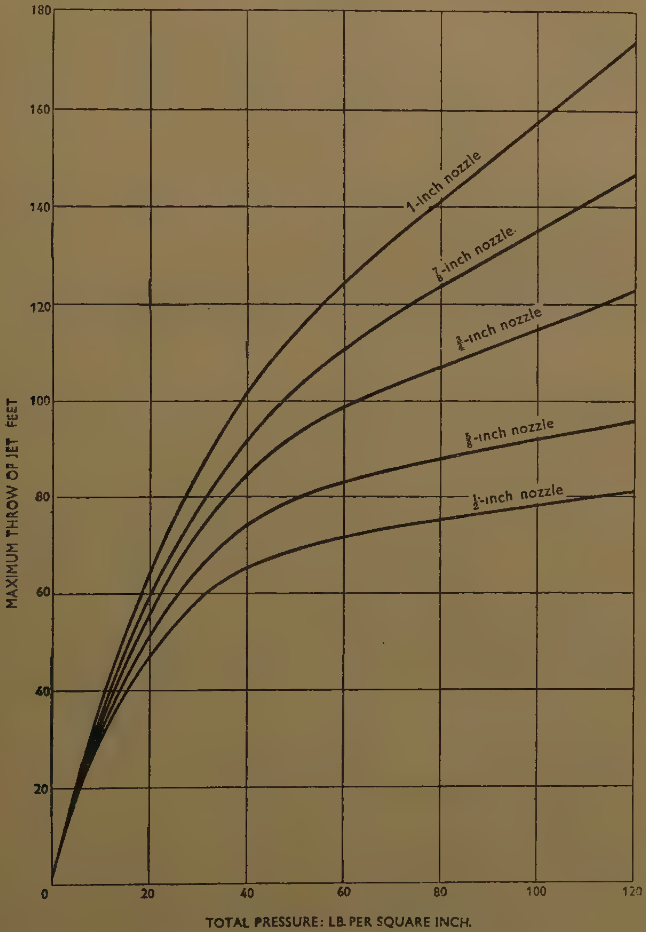
Fig. 15.



The most important information is, therefore, not the exact value of the throw for any given pressure or angle, but the relative throw at different angles and also the relative throw at different pressures. The graphs are also useful, not only in determining the maximum throws, but also in

conjunction with the height-curves, in giving an indication as to whether or not a fire could be reached by projecting the water-stream over an obstacle. Moreover, these graphs enable the throw to be determined in the special case of fighting a fire in a confined space where it is impossible

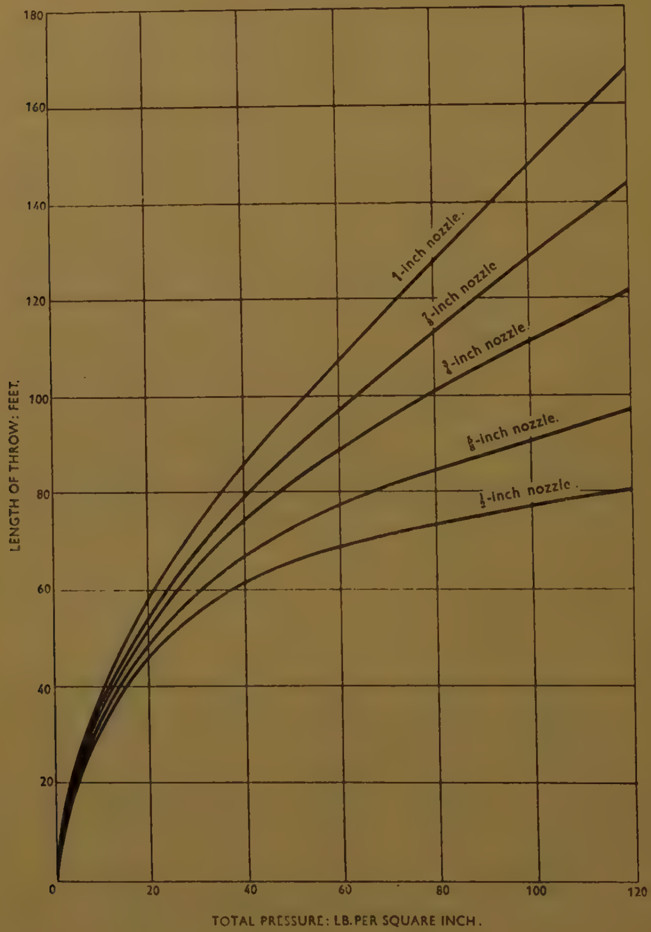
Fig. 16.



to project the jet more than a certain height; for example, in a corridor or a mine-roadway. *Fig. 17* illustrates the throws for a maximum rise of 5 feet above the nozzle. It must be remembered that these throws refer to the total distance at the level of the nozzle and that if, for example, a fire were being fought in a mine-roadway, only half this throw would be effective, since it might be necessary to reach the roof of the roadway, which, in this case, is assumed to be 5 feet above the nozzle.

It will be seen that if the pressure at the nozzle can be maintained, a considerable increase of throw is obtainable by increasing the size of the jet. On the other hand, in most cases where water is being delivered through long pipe-lines or through long hoses, an increase in jet-size so

Fig. 17.



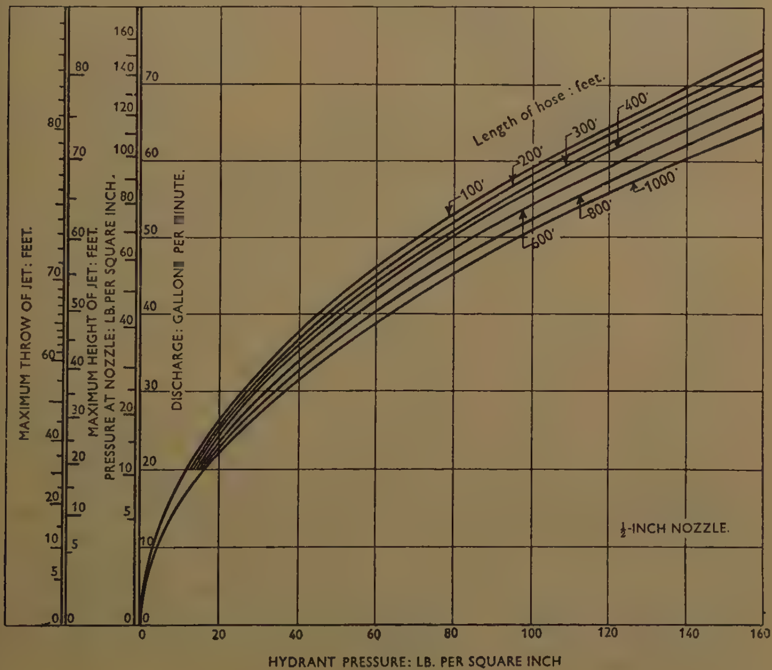
increases the velocity in the pipe or hose that the pressure at the nozzle is reduced very considerably. In such cases a greater throw may be rendered available by decreasing the nozzle size so as to improve the pressure at the nozzle at the expense of a rather smaller quantity of water. It is in situations such as these that the adjustable nozzle is particularly effective, since it is possible to stand at a suitable distance from the fire and then, by

closing or opening the nozzle, to adjust it to such a size that the jet plays effectively on the fire.

Utilization of the Foregoing Data.

Whilst the data given above deal separately with each item affecting the flow of water from fire jets, it is the overall effect of all items together which is usually required; that is, it may be necessary to know the dis-

Fig. 18.



charge, height, and throw of a given nozzle when attached to a certain length of hose and with definite pressures at the hydrant or fire-pump. The step-by-step evaluation of these factors is rather tedious, although, of course, the data enable this to be done. *Figs. 18–22* show the various factors gathered together on one graph for each size of nozzle, and from these graphs it is possible to evaluate the discharge, the maximum throw, and the maximum height of the jet, given the hydrant-pressure, the length of hose, and the size of nozzle. The nozzle-pressure is also indicated, and from a knowledge of this the height and throw at any angle can be derived from *Figs. 5–9* and *11–15*. In these graphs the value given as the hydrant- or pump-pressure is the value while the actual flow is taking place, and not

Fig. 19.

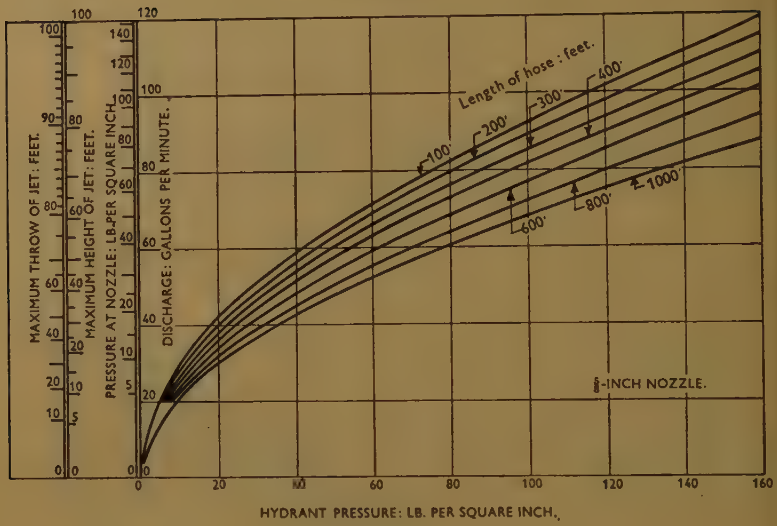
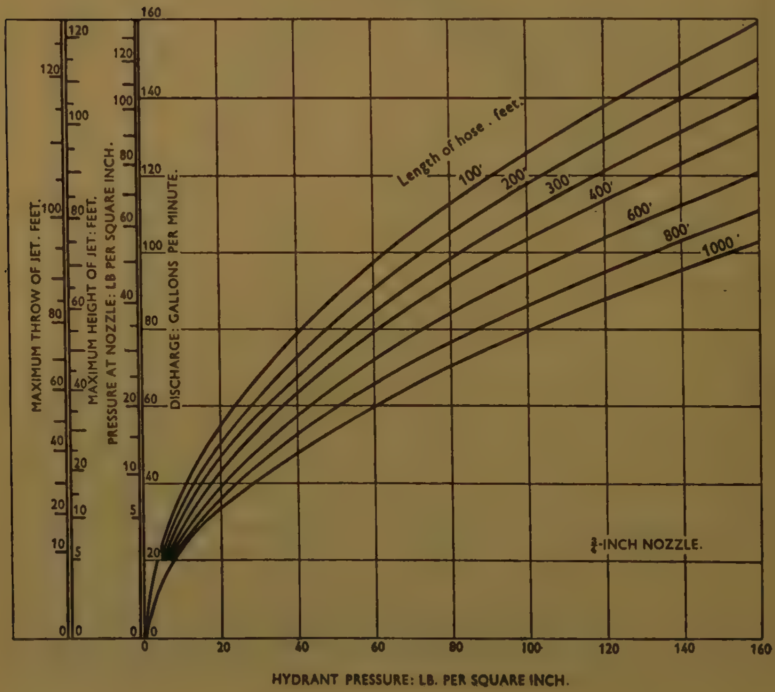

MAXIMUM THROW OF JET: FEET.
MAXIMUM HEIGHT OF JET: FEET.
PRESSURE AT NOZZLE: LB. PER SQUARE INCH.
DISCHARGE: GALLONS PER MINUTE.
Length of hose : feet.
100'
200'
300'
400'
600'
800'
1000'
$\frac{5}{8}$-INCH NOZZLE.
HYDRANT PRESSURE: LB. PER SQUARE INCH.

Fig. 20.


MAXIMUM THROW OF JET. FEET.
MAXIMUM HEIGHT OF JET: FEET.
PRESSURE AT NOZZLE: LB PER SQUARE INCH.
DISCHARGE: GALLONS PER MINUTE.
Length of hose . feet.
100'
200'
300'
400'
600'
800'
1000'
$\frac{3}{4}$-INCH NOZZLE.
HYDRANT PRESSURE: LB. PER SQUARE INCH.

the static head. The graphs can also be used in the inverse manner, to calculate the requisite hydrant-pressure, from a knowledge of other requirements.

Fig. 21.

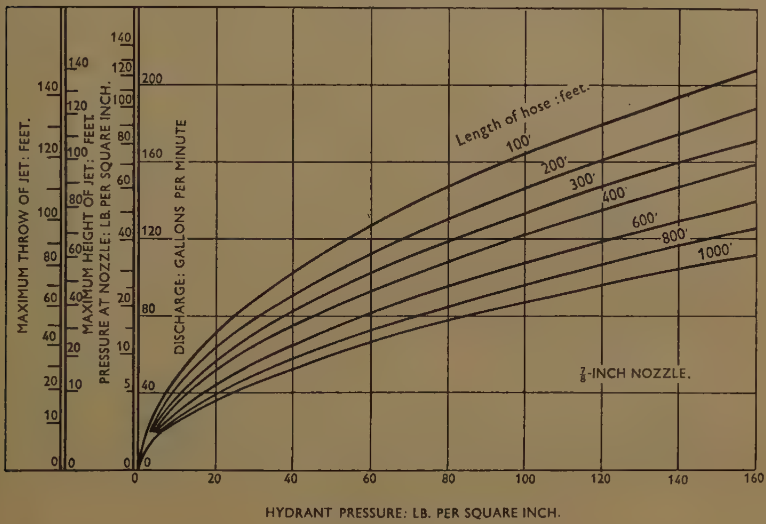
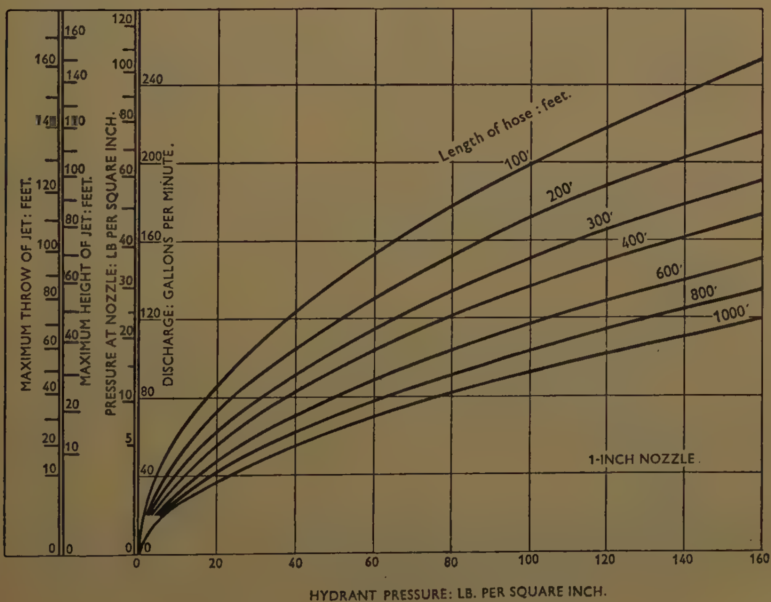


Fig. 22.



Conclusions.

Nozzle Efficiency and Discharge.

The efficiency of modern straight-tapered fire-nozzles is very high, ranging between 96 and 100 per cent. for sizes from ½ inch to 1 inch. Adjustable nozzles are about 5 per cent. less efficient than are the corresponding sizes of plain nozzle. The discharge from a fire nozzle under a pressure of P lb. per square inch can be calculated from the formula :—

$$G = 24{\cdot}7 E_d \, d^2 \sqrt{P},$$

where G denotes the discharge, in gallons per minute ; E_d the discharge efficiency ; d the diameter of the jet, in inches ; and P the total pressure, in lb. per square inch. This formula is sufficiently accurate to form a convenient basis for the estimation of the quantity of water discharged from hydrants, fire pumps, etc.

Frictional Losses in Fire Hose.

For unlined canvas hose between 2-inch and 3-inch diameter, the following expression may be used for the pressure-loss in the hose, couplings, etc., when laid in normal curves on the ground :—

$$p = \frac{G^{1{\cdot}85}}{4{\cdot}33 d^5},$$

where p denotes the pressure-drop, in lb. per square inch per 100 feet of hose, and G and d have the values given above.

Thin-rubber-lined hose will have a pressure-drop of approximately 60 per cent. of that for unlined canvas hose, and thick-rubber-lined hose 45 per cent.

Heights of Jets.

The relationship between height, pressure, angle of projection, and size of jets, is illustrated in *Figs. 5–10.*

Throws of Jets.

The relationship between throw, pressure, angle of projection, and size of jet is illustrated in *Figs. 11–16.*

All of the tests described refer to nozzles of between ½-inch and 1-inch bore, and to pressures of less than 140 lb. per square inch, and the various graphs and values given must not be extrapolated so as to apply outside those limits.

The Paper is accompanied by twenty-eight sheets of drawings, from some of which the Figures in the text have been prepared, by one photograph, and by the following Appendix.

APPENDIX.

Examples to Show the Use of Various Graphs.

Example 1.

Given hydrant-pressure 150 lb. per square inch, 1,000 feet of $2\frac{1}{2}$-inch unlined canvas hose, and $\frac{7}{8}$-inch nozzle, it is required to find the nozzle-pressure, the discharge in gallons per minute, the maximum height and throw, and the height and throw at 40 degrees.

1*st Method*—by trial and error :—The total pressure-loss of 150 lb. per square inch will be made up of the hose-loss plus the nozzle-pressure: inspection of *Fig. 3* shows that the discharge must be less than 125 gallons per minute; therefore, assuming 90 gallons per minute, the pressure-loss in 1,000 feet of hose is 80 lb. per square inch. *Fig. 1* gives 23 lb. per square inch pressure at a $\frac{7}{8}$-inch nozzle for 90 gallons per minute, so that the total pressure-loss is 103 lb. per square inch, which is too low. Next, therefore, assuming 110 gallons per minute,

the pressure-loss in 1,000 feet of hose is	117 lb. per square inch	
the pressure at the nozzle is	35 „ „ „ „	
Total:	152 „ „ „ „	, which is near enough.

The heights and throws can be obtained from *Figs. 8, 10, 14,* and *16,* and are as follows :—

Maximum height	60 feet
Height at 40 degrees	24 „
Maximum throw	83 „
Throw at 40 degrees	75 „

2nd Method. The use of *Fig. 21* avoids trial-and-error methods and gives the following values directly :—

Gallons per minute	109
Nozzle-pressure	35 lb. per square inch.
Pressure-loss in hose	115 „ „ „ „
Maximum height	60 feet.
Maximum throw	83 „

The height and throw at 40 degrees are obtained from *Figs. 8, 10, 14,* and *16,* as before.

Example 2.

This, and the following two examples, show the effect of changing the size of nozzle; the hydrant pressure and the length of hose are the same as in Example 1, but a 1-inch jet is assumed instead of $\frac{7}{8}$-inch jet.

From *Fig. 22*

Discharge	115 gallons per minute.
Nozzle pressure	22 lb. per square inch.
Loss in hose	128 „ „ „ „
Maximum height	44 feet.
Maximum throw	68 „

thus, increasing the size of the nozzle reduces height and throw.

Example 3.

If, however, the nozzle pressure had been maintained at 35 lb. per square inch, the discharge would have been 148 gallons per minute; the maximum height would have been 66 feet; and the maximum throw would have been 94 feet. This would necessitate a hydrant-pressure of 235 lb. per square inch, which is, of course, too great for the usual type of hose.

Example 4.

If, on the other hand, the nozzle be reduced to $\frac{3}{4}$ inch, *Fig. 20* gives the following values :—

Discharge	99 gallons per minute.
Nozzle pressure	52 lb. per square inch.
Loss in hose	98 ,, ,, ,, ,,
Maximum height	72 feet.
Maximum throw	93 ,,

that is, practically the same maximum height and throw as would be obtained by maintaining the 1-inch nozzle pressure at 35 lb. per square inch, although, of course, the discharge is about 33 per cent. less.

Example 5.

It is required to find the hydrant pressure to give an 80-foot throw at 15 degrees with a $\frac{7}{8}$-inch nozzle and 400 feet of hose.

From *Fig. 13,* the nozzle pressure to give an 80-foot throw at 15 degrees is 50 lb. per square inch.

From *Fig. 20,* the hydrant pressure required with 400 feet of hose is 88 lb. per square inch, that is, the loss in the hose is 38 lb. per square inch.

Example 6. *Use of Twin-Hose.*

In many cases the use of twin-hose will be advantageous, particularly in cases where pumping lines are very long. The two hoses are laid side by side and joined together by a Y-piece near the nozzle end, so as to combine the discharge from the two hoses in the one nozzle. Owing to the smaller velocity in each hose by this arrangement in comparison with a single hose, the friction loss is considerably reduced, as will be seen from the following example, which should be compared with Example 1, which gives the corresponding case for single hose.

Hydrant pressure	150 lb. per square inch.
Hose	1,000 feet of $2\frac{1}{2}$-inch.
Nozzle	$\frac{7}{8}$ inch.

Case (*a*). Single hose.	Discharge	109 gallons per minute.
	Nozzle pressure	35 lb. per square inch.
	Loss in hose	115 ,, ,, ,, ,,
	Maximum height	60 feet.
	Maximum throw	83 ,,

Case (*b*). Twin hose. (By trial and error.)

The total loss in the hose will be approximately two-thirds of that for one hose, say 76 lb. per square inch : therefore,

Nozzle pressure	= 74 lb. per square inch.
Discharge	=162 gallons per minute.
Flow in each hose	= 81 ,, ,, ,,
Loss	= 68 lb. per square inch.

This is too small, so, assuming 72 lb. per square inch for hose-loss,

Nozzle pressure	= 78 lb. per square inch.
Discharge	=166 gallons per minute.
Flow in each hose	= 83 ,, ,, ,,
Loss	= 70 lb. per square inch.

which is near enough.

In other words, doubling the hose line has increased

the discharge from	109 to 166 gallons per minute,
the nozzle pressure from	35 to 78 lb. per square inch,
the maximum height from	60 to 104 feet,
and the maximum throw from	83 to 122 ,,

Paper No. 5259.

"Some Experiments on the Consolidation of Clay."

By Leonard Frank Cooling, M.Sc., and Alec Westley Skempton, M.Sc., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

	PAGE
Introduction	381
The oedometer test	383
Theory of consolidation	386
Description of experiments	388
Conclusions	396
Acknowledgements	398

Introduction.

The consolidation of clays was first studied by Professor Karl von Terzaghi, M. Inst. C.E.[1], and his theory of consolidation has proved of great value in problems of foundation design involving settlement[2].

The term "consolidation", as used in soil mechanics, refers to the process by which a soil undergoes a decrease in volume when it is subjected to an increase in pressure. This volume-reduction, or compression, is brought about by a closer packing of the grains, and its magnitude depends on the pressure-increment and on the compressibility of the soil. With fine-grained soils such as clays and silts, the voids of which are completely filled with water, the decrease in volume follows only gradually on the application of pressure, and a considerable time elapses before the soil reaches a state of equilibrium.

It was to explain this time-lag that Terzaghi advanced his theory of consolidation. He pointed out that as the pores are completely filled with water, a volume-reduction can occur only by the escape of a corresponding volume of water from the clay into a free drainage surface. He suggested that the mechanism of the process was as follows. The pressure-increment is first taken wholly by the pore water, thus setting up in the pore water an excess hydrostatic pressure. At a drainage surface the pore water has zero excess pressure, and consequently a hydraulic gradient is set up, which in turn causes water-movements. The consolidation process is then governed by the gradual dissipation of the excess

[1] "*Erdbaumechanik*," Chapters 12, 20, and 21. Deuticke, Vienna, 1925.

[2] "Soil Mechanics. A New Chapter in Engineering Science." Journal Inst. C.E., vol. 12 (1938–39), p. 106 (June 1939).

hydrostatic pressure and, as fine-grained soils offer a high resistance to the flow of water, the process will be slow. When the excess pressure has finally become equal to zero the soil is in equilibrium under the applied pressure. This pressure is then said to be " effective."

In the mathematical development of his theory, Terzaghi made the simplifying assumption that the flow of water is one-dimensional only, a condition which would obtain for a layer of clay lying between two sand layers, or between one sand layer and a bed of shale or rock. He was then able to show that the consolidation process could be represented by a partial differential equation of the second order, similar in form to the Fourier law of linear diffusion, and involving a constant for the material termed the " coefficient of consolidation." The solution of this equation requires that the position of the drainage surfaces and the pressure-distribution in the layer should be known. For a full treatment of this analysis reference should be made to the standard work by Terzaghi and Fröhlich[1]. In the present Paper only the results of the analysis will be outlined.

Terzaghi also carried out a series of tests to study the extent to which the theory represents the actual consolidation process in clays. For this purpose he developed the " oedometer ", an apparatus in which a clay layer was subjected to a load under conditions similar to those postulated in the theory, that is, resulting in a one-dimensional flow of water. For the clays with which he was working, he was able to show that the experimental time/consolidation curves were of the same general shape as the theoretical curves, except that they showed some deviation in the later stages of consolidation[2]. He also stated, however, that with some soils the theory was not closely followed.

Since that time many oedometer tests have been made, but very few accounts of comparisons between experimental and theoretical results have been published. The general conclusion of the Harvard laboratory[3], in 1936, was that the deviations in the later stages of consolidation occur to a greater or less degree in all soils, but are most pronounced in soils containing organic matter.

In this Paper a brief outline is given of the oedometer test, with descriptions of a few simple experiments on samples prepared from a stock of London clay made homogeneous by remoulding with water. The tests serve to show that London clay yields results in close agreement with theory, and also demonstrate in a straightforward manner some of the main conclusions of the theory.

---

[1] K. von Terzaghi and O. K. Fröhlich, " *Theorie der Setzung von Tonschichten.*" Deuticke, Vienna, 1936.

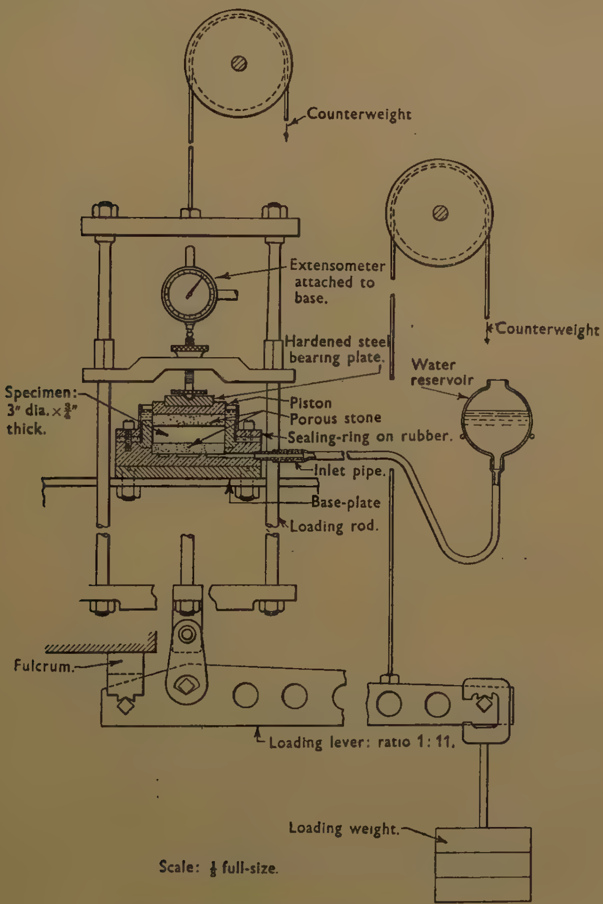
[2] " Principles of Final Soil Classification." *Public Roads*, vol. 8 (1927), p. 41 (May 1927).

[3] H. Gray, " Progress Report on Research on the Consolidation of Fine-Grained Soils." Proc. Int. Conf. on Soil Mechanics, Harvard University, 1936. Vol. II, Paper D. 14.

The Oedometer Test.

A modern form of oedometer, developed by Terzaghi and Casagrande, is illustrated diagrammatically in *Fig. 1.* The sample of clay is held in a brass ring between porous stones, which provide a free drainage surface

Fig. 1.



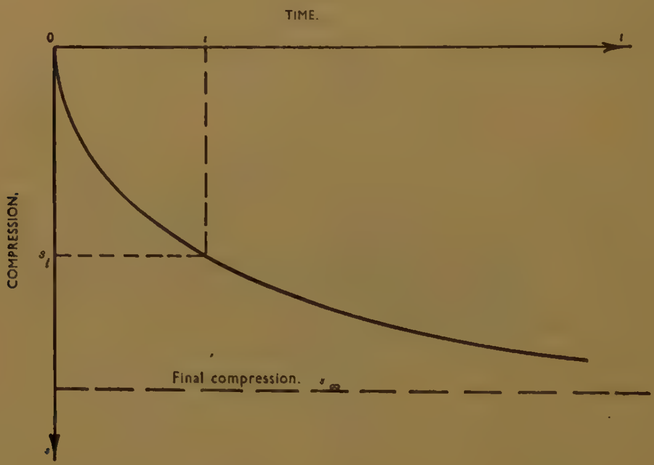
Consolidation Test Apparatus.

for the water expelled from the sample during consolidation. The stones consist of disks about ½ inch thick, of an open-pored Portland limestone, and they are kept in contact with water to prevent the sample from drying out during the test. The samples used in a standard test are 3 inches in diameter and ¾ inch thick, but oedometers of 4-inch diameter, taking

samples of variable thickness, are also in use. Pressure is applied to the sample through the upper porous stone by placing a weight on the hanger at the end of a lever-arm. The lever system is counterbalanced so that very small pressures can be used, and is arranged so that a weight of 10 lb. on the hanger applies a pressure of 1 ton per square foot on the sample.

Since the sample is restrained laterally, the consolidation process can be followed by observing its decrease in thickness. This is done by means of a dial gauge reading to 1×10^{-4} inch. On applying a pressure, compression takes place, rapidly at first, but the rate decreases gradually with time, as shown in *Fig. 2*.

Fig. 2.



After an interval, largely depending upon the thickness of the sample, the dial movements practically cease, and the sample can then be considered to be in equilibrium.

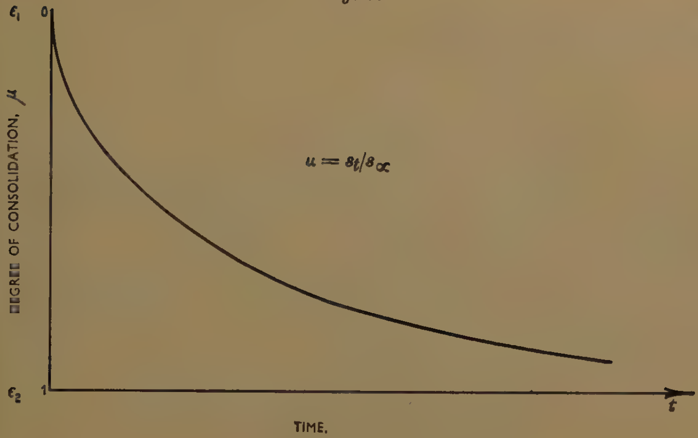
The final compression of the sample under this pressure is denoted by s_∞ (since theoretically an infinite time is required for equilibrium to be reached) and the compression at any time t after application of the pressure is denoted by s_t. The degree of consolidation μ at this time is defined by the equation

$$s_t = \mu \, . \, s_\infty,$$

and the s/t curve of *Fig. 2* can readily be plotted as a μ/t curve, as in *Fig. 3*.

After an approximately steady state has been reached the pressure is increased, when a similar consolidation process occurs and a new equilibrium-density of the clay under the increased pressure is attained.

Fig. 3.

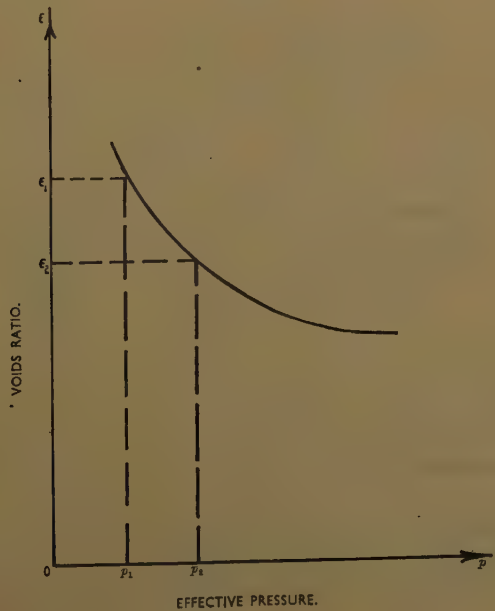


Successive pressure-increments are applied, and give a corresponding number of equilibrium-densities. The density of a sample is expressed as the voids ratio, defined by the equation

$$\epsilon = \frac{\text{volume of voids}}{\text{volume of solids}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and the relation between effective pressure and voids-ratio (the ϵ/p curve) is shown in *Fig. 4.*

Fig. 4.



The oedometer test therefore gives one pressure/voids-ratio curve and a number of time/consolidation curves.

Theory of Consolidation.

The p/ϵ curve is a characteristic of the clay, and from it the compressibility can be determined. The compressibility, for linear consolidation, is defined as the decrease in thickness per unit thickness per unit pressure-increase, and can be determined in the following manner.

A clay sample has an original thickness l_1 and a voids ratio ϵ_1, when in equilibrium under a pressure p_1. If the pressure be increased by σ the sample consolidates to a final thickness l_2 and voids ratio ϵ_2 under the pressure $p_2 = p_1 + \sigma$. Now from equation (1),

$$\frac{\text{original volume of voids}}{\text{original volume of sample}} = \frac{\epsilon_1}{1 + \epsilon_1},$$

and therefore
$$\frac{\text{decrease in volume of voids}}{\text{original volume of sample}} = \frac{\epsilon_1 - \epsilon_2}{1 + \epsilon_1};$$

but in the test the decrease in volume equals the decrease in volume of voids: hence

$$\frac{\text{decrease in volume of sample}}{\text{original volume of sample}} = \frac{\epsilon_1 - \epsilon_2}{1 + \epsilon_1};$$

and as the area of the sample remains constant,

$$\frac{\text{decrease in thickness}}{\text{original thickness}} = \frac{\epsilon_1 - \epsilon_2}{1 + \epsilon_1},$$

or
$$l_1 - l_2 = \frac{\epsilon_1 - \epsilon_2}{1 + \epsilon_1} \cdot l_1 \qquad . \; . \; . \; . \; . \; . \; . \; (2)$$

The compressibility is, therefore, given by the equation

$$\text{compressibility} = \frac{l_1 - l_2}{\sigma \, . \, l_1} = \frac{\epsilon_1 - \epsilon_2}{\sigma(1 + \epsilon_1)}.$$

Denoting the average slope of the p/ϵ curve over the increment σ by a,

$$a = \frac{\epsilon_1 - \epsilon_2}{\sigma},$$

or in the limit,
$$a = -\frac{d\epsilon}{dp},$$

and the compressibility
$$= \frac{a}{1 + \epsilon_1}.$$

The compressibility is not, in general, a constant, but is a function of ϵ, and hence of p. From the p/ϵ curve the value of $\frac{a}{1 + \epsilon}$ can, therefore,

be calculated for any value of ϵ or p, and the p/ϵ curve also enables the final compression of a clay layer to be calculated for any pressure increment, from equation (2).

The rate at which the consolidation proceeds is, however, a more complicated problem, and it is to this problem that the theory of consolidation is applied. The principal result of this theory is that the degree of consolidation μ is a function only of the factor (ct/d^2).

In this factor c is the "coefficient of consolidation", a constant for a given clay, t denotes the time after application of the load causing consolidation, and d denotes the "drainage path", that is, the maximum distance which water has to travel in the clay (measured in a straight line) before reaching a free drainage-surface. In the oedometer test d is one-half the thickness of the sample, as the water can drain from both faces into the porous stones.

The above result may be written in the form

$$\mu = \psi\left(\frac{ct}{d^2}\right) = \psi(\tau) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where τ denotes the "time factor", which is dimensionless, and c has the dimensions $L^2 . T^{-1}$.

The function ψ depends upon the distribution of pressure in the layer and upon whether one or both faces of the layer constitute drainage-surfaces. For the particular conditions of the oedometer test, namely, drainage from both faces of the layer and a uniform distribution of pressure, the solution is

$$\mu = 1 - \frac{8}{\pi^2}\left[e^{-\frac{\pi^2}{4}\tau} + \frac{1}{9}e^{-\frac{9\pi^2}{4}\tau} + \frac{1}{25}e^{-\frac{25\pi^2}{4}\tau} + \ . \ . \ . \right] \quad . \quad . \quad (4)$$

Values of μ calculated from the above equation for some particular values of τ are given in the following Table :—

τ	μ	τ	μ	τ	μ
0	0	0·200	0·504	0·800	0·887
0·008	0·101	0·300	0·613	1·00	0·931
0·020	0·160	0·400	0·698	2·00	0·994
0·048	0·247	0·500	0·764	∞	1·000
0·100	0·357	0·600	0·816		

Now it follows from equation (3) that, for any given conditions of pressure and drainage, and for a particular clay, the value of μ at any given time varies inversely as d^2. In order to confirm this result in a simple manner, oedometer tests were carried out on three samples of the same clay with different thicknesses.

A further check on the degree of validity of the theory is rendered possible by comparing the shape of the experimental time/consolidation

curves with the curve defined by equation (4). This is the comparison which was referred to in the introduction. As will be seen later, the comparison is made a relatively simple matter by using the result that in the range $0 < \mu < 0{\cdot}5$, μ is proportional to $\sqrt{\tau}$. This can be found from equation (4), but it has been shown more directly by Dr. E. N. Fox, of the Building Research Station, who has derived the equation

$$\mu = \frac{2}{\sqrt{\pi}}\sqrt{\frac{ct}{d^2}} = 2\sqrt{\frac{\tau}{\pi}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

This equation is true to a high degree of accuracy within the range indicated, and was obtained by the method of the operational calculus, as applied by Dr. H. Jeffreys [1] to problems in the conduction of heat, which are analogous mathematically to those in the consolidation of clays.

Description of Experiments.

Material used in experiments.

In order to carry out the experiments with material as homogeneous as possible, about 1 cubic foot of London clay was remoulded very thoroughly with water to form a stiff slurry and was allowed to stand for several weeks under damp cloths in order further to equalize the water-content distribution. From this mass of clay small samples were taken, and on these samples various simple tests, principally for classification purposes, were carried out. For the methods of carrying out these tests, and for their physical significance, reference should be made to a recent text-book [2] on soil mechanics. Very little variation was found to exist between the different samples, and the mean results were as follows :—

Water-content: per cent. of dry weight	76
Liquid limit: per cent. of dry weight	78
Plastic limit: per cent. of dry weight	25
Shrinkage limit: per cent. of dry weight	15
Specific gravity of particles: per cent. of dry weight	2·71

		Millimetres.	Per cent.
Mechanical analysis	Coarse sand .	2·0–0·2	0
	Fine sand . .	0·2–0·02	31
	Silt	0·02–0·002	21
	Clay	< 0·002	46
	Loss on H_2O_2 and HCl pre-treatment		2

The above values indicate a typical inorganic clay.

[1] H. Jeffreys, "Operational Methods in Mathematical Physics." Cambridge Tracts in Mathematics and Mathematical Physics, No. 23 (1931), Chapter 5.

[2] F. L. Plummer and S. M. Dore, "Soil Mechanics and Foundation Engineering." Pitman, New York, 1940.

Experimental procedure.

The clay slurry was placed in three oedometers, one of 3-inch diameter and two of 4-inch diameter, until the thickness of clay was about 1·0 inch, 1·6 inch, and 3·1 inches respectively. These samples will be referred to as Samples I, II, and III. The clay was gently tamped into position, to exclude as far as possible any small air-bubbles, although it was not to be expected that they could be avoided entirely.

The upper porous stone was then placed on the surface of the clay, which had been carefully smoothed off, and a pressure of about 0·1 ton per square foot was applied through the piston. The dial-gauge showed that the clay at once started consolidating, although part of the movement under this load was due to a slight extrusion of slurry into the space between the stone and the brass ring. After the sample had been subjected to this pressure for 1 day, the pressure was increased to 0·5 ton per square foot, which was maintained until the dials showed that the rate of movement had become sensibly zero. With Sample I this occurred after 4 days, whilst 10 days were allowed for Samples II and III, the movements during the last 24 hours being 3×10^{-4} inch, 3×10^{-4} inch, and 8×10^{-4} inch respectively. Under this pressure the clay became quite firm.

The pressure was then successively increased on each sample from 0·5 ton per square foot to 1 ton, 2 tons, and 4 tons per square foot, each increment being maintained for 3, 5, and 9 days on Samples I, II, and III. Dial readings were taken throughout: the final compressions under each increment are given in Table I. The actual observations for the three samples under the increment from 1 ton per square foot to 2 tons per square foot are plotted in *Fig. 5*.

The average compressions during the last 24 hours of each increment were 2×10^{-4} inch, 5×10^{-4} inch, and 1×10^{-4} inch for the three samples, or less than 1 per cent. of the final compression movements.

When each sample had come to this approximate equilibrium under the greatest pressure of 4 tons per square foot, the load was removed, the oedometer was dismantled, and the sample was weighed (W_f); it was then placed in a ventilated oven maintained at 105° C. and periodically reweighed, after being allowed to cool in a desiccator. When a constant weight W_0 had been reached, it was assumed that all the water had been driven off, and the water-content of the sample when in equilibrium under 4 tons per square foot was then determined from the relation

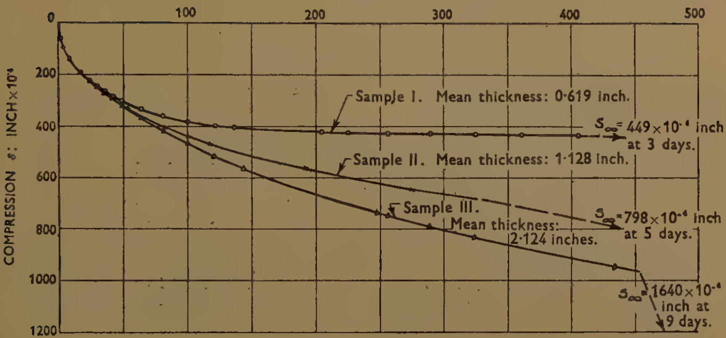
$$w_f = \frac{W_f - W_0}{W_0} \times 100.$$

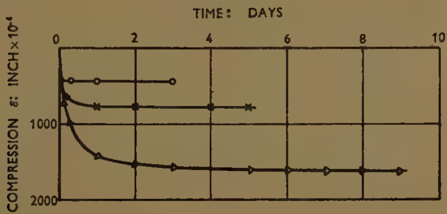
The results of 33·7, 33·8, and 33·4 for samples I, II, and III, are probably fortuitously close, although a reasonable agreement would be expected on account of the homogeneity of the material.

TABLE I.

Effective pressure, p : tons per square foot.	Sample I.			Sample II.			Sample III.		
	Final compression, s_∞ : inch.	Voids ratio, ϵ.	Thickness of sample, l : inch.	Final compression, s_∞ : inch.	Voids ratio, ϵ.	Thickness of sample, l : inch.	Final compression, s_∞ : inch.	Voids ratio, ϵ.	Thickness of sample, l : inch.
0·5	0·3	1·329	0·680	0·4	1·316	1·226	0·7	1·357	2·362
1·0	0·0389	1·196	0·641	0·0582	1·206	1·168	0·1560	1·201	2·206
2·0	0·0449	1·041	0·596	0·0798	1·055	1·088	0·1640	1·037	2·042
4·0	0·0373	0·914	0·559	0·0736	0·916	1·014	0·1320	0·905	1·910

Figs. 5.





The pressure/voids-ratio curve.

From the definition of voids-ratio given in equation (1) it follows that, if the voids are saturated with water, the value of ϵ under 4 tons per square foot is given by

$$\epsilon_f = \rho \cdot \frac{w_f}{100} \text{ (where } \rho \text{ denotes the specific gravity of the particles).}$$

The three values are plotted in *Fig. 6.* (The slight differences in pressure are due to the pressure on all three samples not being exactly equal, but the differences are negligibly small.)

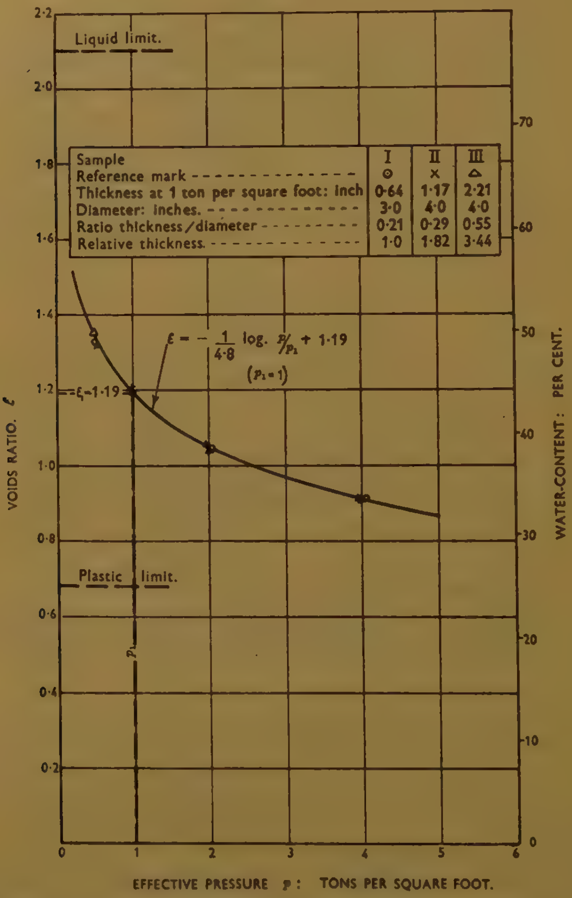
The thickness of the samples l_f under this pressure can also be calculated, for if A denote the area of the sample and γ_w the density of water, then

$$W_f = A \cdot l_f \cdot \frac{\epsilon_f + \rho}{1 + \epsilon_f} \cdot \gamma_w$$

These values are given in Table I, and from the observed final compressions it is at once possible to calculate the thickness under the other pressures. It is also possible, with the aid of equation (2), to calculate the values of ϵ corresponding to the changes in thickness; the results of *Fig. 6* were obtained in this way. From *Fig. 6* and from Table I it is seen that each sample has practically the same voids-ratio under any

given pressure, which indicates that the thickness of the sample has little effect upon the p/ϵ curve.

Fig. 6.



In his original research on the consolidation of clays, von Terzaghi found that the relation between voids-ratio and pressure could be expressed to a good approximation by the equation

$$\epsilon = -\frac{1}{B}\log\frac{p}{p_1} + \epsilon_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where B is a constant and ϵ_1 denotes the voids-ratio under a pressure p_1 of unity. The experimental points lie on this equation, the values of the constants being $\epsilon_1 = 1{\cdot}19$ and $B = 4{\cdot}8$.

The compressibility at any pressure p is readily found from equation 6), as

$$a = -\frac{d\epsilon}{dp} = \frac{1}{Bp},$$

nd the value of ϵ, which enables $\frac{a}{1+\epsilon}$ to be evaluated, can be taken rom the p/ϵ curve or from equation (6).

The time/consolidation curves.

In order to check the theoretical result that the value of μ at any ime t varies inversely as d^2, the results of *Fig. 5* have been replotted in *Fig. 7*, showing the relation between μ and t/d^2. It will be seen from this hat within close limits the theoretical result is confirmed, as was also the ase with the other two pressure-increments.

In studying the general shape of the time-consolidation curves, one of he most convenient methods is to plot μ against $\sqrt{t}$, as first suggested by Drs. Gilboy and Taylor[1]. The first half of the relation should then be epresented by a straight line; and a typical $\mu/\sqrt{t}$ curve, given in *Fig. 8*, hows that for the clay used in these experiments this is true.

From the slope of the linear portion of the $\mu/\sqrt{t}$ curve the value of c an be calculated, and the full theoretical curve can then be plotted from quation (4). Thus, in *Fig. 8*, the slope of the linear portion is

$$(\mu/\sqrt{t})_{\text{experimental}} = \frac{1}{10{\cdot}1} \text{ minute}^{-\frac{1}{2}},$$

nd the value of d, which is one-half the mean thickness of the sample, is

$$d = \tfrac{1}{2} \times 0{\cdot}619 = 0{\cdot}310 \text{ inch}.$$

rom equation (5) it is known that

$$(\mu/\sqrt{t})_{\text{theoretical}} = 2\sqrt{c/\pi d^2};$$

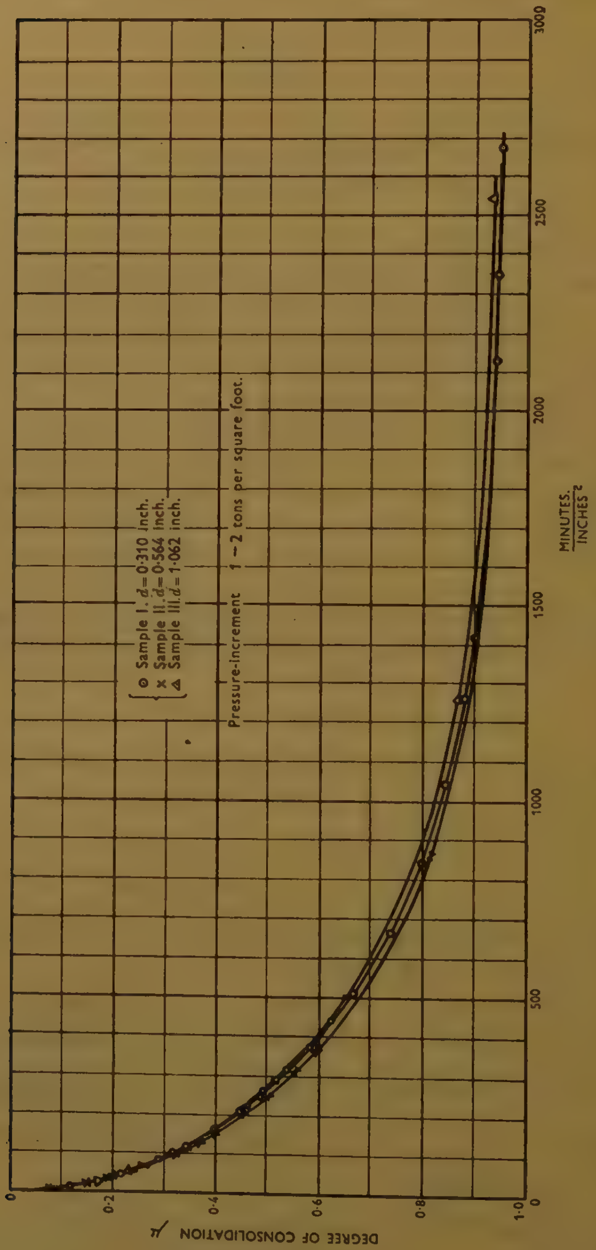
nd hence, for equal slopes of the linear portions of the experimental and heoretical curves, the value of c is

$$c = \frac{\pi}{4} \cdot \frac{d^2\mu^2}{t} = \frac{\pi}{4} \times \frac{0{\cdot}310^2}{10{\cdot}1^2} = 7{\cdot}4 \times 10^{-4} \frac{(\text{inches})^2}{\text{minutes}}.$$

In American laboratories it is usual to express the coefficient of conolidation in $\frac{(\text{centimetres})^2}{\text{seconds}}$. In these units the above value equals $\cdot 0 \times 10^{-5} \frac{(\text{centimetres})^2}{\text{seconds}}$.

[1] G. Gilboy, "Improved Soil Testing Methods." *Engineering News-Record*, ol. 116 (1936), p. 732 (21 May, 1936).

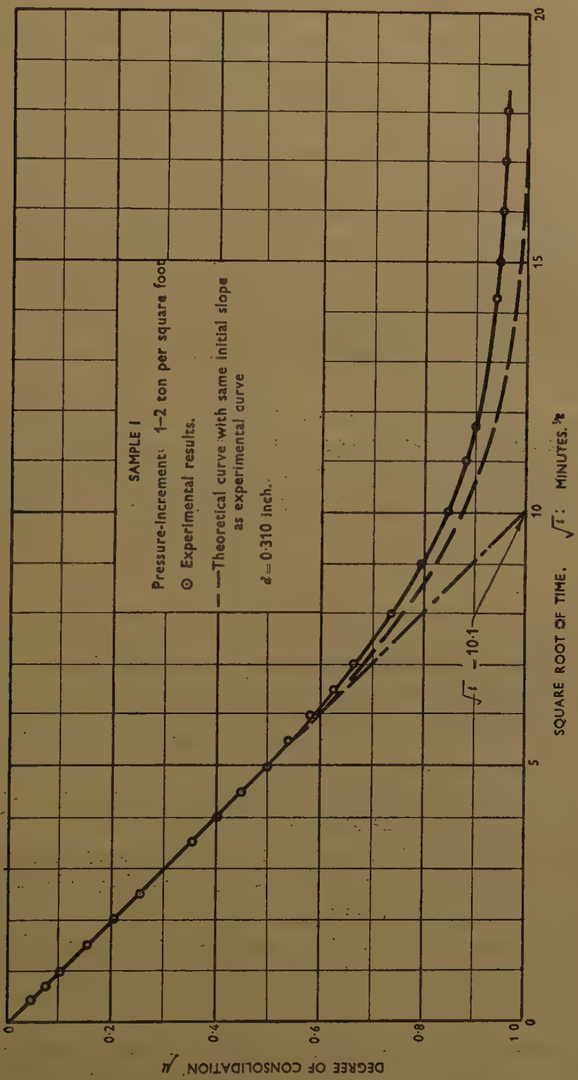
Fig. 7.



⊙ Sample I. d = 0·310 inch.
× Sample II. d = 0·564 inch.
△ Sample III. d = 1·062 inch.
Pressure-increment 1 — 2 tons per square foot.
DEGREE OF CONSOLIDATION μ
0
0·2
0·4
0·6
0·8
1·0
0
500
1000
1500
2000
2500
3000
MINUTES. / INCHES²

The full theoretical curve can now be plotted, as shown dotted in *Fig. 8*, and this is seen to deviate slightly from the experimental curve in the later stages of consolidation, as mentioned in the introduction. This

Fig. 8.



deviation is known as the "secondary time effect" or as the "secondary compression" and, according to Drs. D. W. Taylor and W. Marchant, it "may be of the nature of a plastic adjustment of the clay structure

wherein all or practically all the shearing stresses are eventually dissipated[1]." The mechanism is not completely understood, and will not be discussed further, except to mention that in soils exhibiting considerable secondary compression a correction has to be applied in the calculation of c. The two principal methods of applying this correction are those in use at the Massachusetts Institute of Technology [1] and at Harvard University [2].

With London clay the secondary compression is small, and the simple method of calculating c, described above, can be used without serious error. The values of c determined by this method are given in Table II. They are about 10 per cent. lower than the effective values obtained by applying the corrections, but this difference is of little practical importance, particularly in view of the variations in the experimental values of c.

Within these variations, the Table shows that the coefficient of consolidation is independent of both pressure and thickness of sample. However, other oedometer tests on remoulded London clay, which have been carried out with pressures ranging up to 9 tons per square foot, indicate a small decrease in the coefficient at pressures greater than about 3 tons per square foot.

The results of numerous tests carried out on samples of London clay in its natural state reveal an agreement with theory of the same order as that described above. The value of the coefficient of consolidation is also of the same order, but the compressibility is, of course, very much lower.

Conclusions.

The following conclusions are valid for London clay, and for the thicknesses of sample used :—

(a) the time/consolidation relation agrees with theory within reasonable practical limits,

(b) the rate of consolidation varies inversely as the square of the thickness of the sample,

(c) the coefficient of consolidation is a constant for the material, and within practical limits is independent of the pressure and thickness of the sample,

(d) the pressure/voids-ratio relation is independent of the thickness of the sample.

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[1] D. W. Taylor and W. Marchant, "A Theory of Clay Consolidation accounting for Secondary Compression." J. Math. and Phys., vol. 19 (1940); p. 167 (July 1940).

[2] A Casagrande, "New Facts in Soil Mechanics from the Research Laboratories." *Engineering News-Record*, vol. 115 (1935), p. 320 (5 Sept. 1935).

TABLE II.

Mean effective pressure, p : tons per square foot.	Sample I.		Sample II.		Sample III.	
	Mean thickness, l : inch.	Consolidation coefficient, c : $\frac{(\text{inches})^2}{\text{minutes}}$.	Mean thickness, l : inch.	Consolidation coefficient, c : $\frac{(\text{inches})^2}{\text{minutes}}$.	Mean thickness, l : inches.	Consolidation coefficient, c : $\frac{(\text{inches})^2}{\text{minutes}}$.
0·75	0·660	$5{\cdot}8 \times 10^{-4}$	1·197	$7{\cdot}5 \times 10^{-4}$	2·284	$7{\cdot}4 \times 10^{-4}$
1·50	0·619	7·4 ,,	1·128	8·1 ,,	2·124	7·5 ,,
3·0	0·578	6·5 ,,	1·051	6·6 ,,	1·976	6·2 ,,
		Mean : 6·6 ,,		Mean : 7·4 ,,		Mean : 7·0 ,,

$$\text{Mean value}: c = 7{\cdot}0 \times 10^{-4} \frac{(\text{inches})^2}{\text{minutes}}.$$

Acknowledgement.

The work described was carried out at the Building Research Station of the Department of Scientific and Industrial Research, and the results are published by permission of the Director of Building Research.

The Paper is accompanied by five sheets of drawings, from which the Figures in the text have been prepared.

Paper No. 5245.

"A Method of Estimating the Maximum Possible Silt Deposit Upstream of Dams Constructed in Silt-Carrying Rivers."

By Abdel Aziz Ahmed, *Bey*, D.Sc., Ph.D., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form*[1].)

In dams primarily intended for water-storage the head available for hydro-electric power generation is inherently variable. At the Aswan dam it ranges from 33·5 metres when the reservoir is full to 3 metres during the flood season. The ratio between these limiting heads (11 : 1) is probably too high to permit of running Kaplan turbines at constant speed throughout the entire range of variation. This difficulty may be overcome by designing the turbine to run at two different speeds, for the upper and the lower head respectively. Generation at constant frequency would be accomplished by coupling to each turbine two alternators, each giving the full output, but having different numbers of poles, one alternator being utilized for each speed and the other running idle.

A simpler solution is to raise the water-level in the reservoir during flood to, say, 6·5 metres, which reduces the ratio to about 5 : 1, and therefore renders it possible to design a turbine to run at one constant speed throughout the entire range of head. It has been argued, however, that this procedure may, with the lapse of years, cause silting up of the reservoir, with consequent reduction of its storage capacity.

Extensive experiments have been carried out for the purpose of determining the quantity of silt deposited owing to raising the level in the reservoir by given amounts during the flood season : but the results obtained so far are remarkably inconsistent, and seem to emphasize the difficulty of arriving at reliable and decisive conclusions.

In this Paper an attempt is made to provide a solution for the determination of the boundary conditions, which enables an estimation to be made of the maximum quantity of silt that can possibly be deposited under given hydraulic conditions. This solution is based upon the assumption that natural silt-carrying rivers are, to a certain extent, self-correcting in the matter of silting and scour : periodic changes in the hydraulic conditions of the river, such as are caused by seasonal floods, give rise to alternate silting and scouring of the river-bed, until critical velocities are attained at all points of any given cross section, when

[1] The MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

neither silting nor scour occurs, provided that the new hydraulic conditions are maintained long enough.

This assumption appears to be justifiable, at all events, in respect of the river Nile, as several dams thrown across it have been in operation for many years and have apparently caused no appreciable alteration in the river-bed. The proposition may be enunciated as follows:

"If the water-level in a given stretch of a silt-carrying river be permanently raised or lowered by a given amount (assumed to be small in comparison with the river-depth), the discharge remaining constant, silt will be deposited in the former case and scoured in the latter case, until a critical velocity is attained, at which neither silting nor scour will occur. In the long run the maximum quantity of silt thus deposited or scoured is, theoretically, equal to the increase or decrease in the water-content of that stretch of the river, caused respectively by raising or lowering the water-level."

In the generalized form of the problem stated above, it is assumed that the critical velocity at which no silting takes place is equal to the limiting velocity at which scour begins. This assumption is incorrect in practice, since the scouring velocity is bound to be higher than the silting velocity; but that does not invalidate the nature of the theoretical conclusion, provided that in the application thereof, for the estimation of the degree of silting or scouring, the practical aspects of the problem are taken into account.

In *Fig. 1*, an elementary slice AB, lying in a given cross-section of the river upstream of the dam, has surface-area S and thickness dl. The critical velocity v, at which neither silting or scour takes place, varies from point to point all over the slice-surface; but for any given point it may be assumed to be constant over the thickness dl, and may be expressed by $v = \frac{dQ}{dS}$ where dS is an element of the surface-area and dQ denotes the elementary discharge passing through it.

If now the water-level in the river be raised by a small amount XY, for example, by partially closing the sluice-gates of a dam, thus causing the surface-area of the slice under consideration to increase, the velocities in the various elements of the area will fall momentarily, for two reasons: firstly, the increase in the area of the slice, and secondly, the decrease in the river-gradient. Consequently, silt will be deposited, causing a reduction in the surface-area of the slice, which in turn reacts on the velocities and causes them to rise again. This procedure continues until permanent and stable conditions are established, when the velocities in the various elementary areas attain their critical values at which no silt is thrown off. Since the rise in the water-level is small, the slice under consideration may be assumed to maintain its original configuration, more or less, and consequently the final critical velocities attained will not be appreciably different from their initial values.

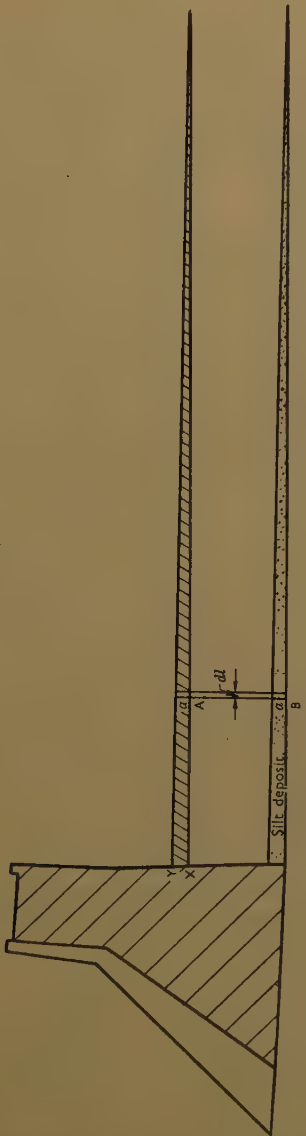

Fig. 1.

It follows, therefore, that when stable conditions are attained, the discharge remaining constant, the total quantity of silt deposited at the bottom of the slice will be of such magnitude as to reduce its area to its original value, and will therefore be equal to $a \times dl$, where a denotes the increase in the slice-area caused by raising the water level, and dl the

thickness of the slice, as before. Hence the total quantity of silt deposited in the entire stretch of the river under study is given by $\Sigma a \,.\, dl$. This is obviously equal to the increase in the water-content of the reservoir caused by raising the water-level.

The argument holds good for any given slice, irrespective of its initial shape and of the critical velocities in it, for, since the general shape of the slice will not be appreciably altered, the critical velocities will finally assume their original values.

This method of reasoning applies also to the case of lowering the level of water in the river instead of raising it, when the consequent scour is similarly equal theoretically to the reduction in the water-content upstream of the dam.

Considering now the application of these theoretical results to practical problems of silting and scour: the limiting velocity at which scour begins is much higher than the critical velocity at which silt is deposited—probably two or three times as high—so that the theoretical critical velocity at which neither silt nor scour takes place must lie somewhere between these two limits. Hence the theoretical quantity of silt deposited or scoured, or its equivalent, the change in the water-content, will, in the case of a rise in level, be greater than the quantity of silt actually deposited, and in the case of a fall in level, smaller than the quantity actually scoured.

Therefore it would be on the safe side to assume the maximum amount of silt-deposit caused by raising the water-level to be equal to the increase in the river-content arising therefrom. Thus, if the level of the Aswan reservoir were raised from X to Y, and maintained there during all flood seasons to come, then the maximum amount of silt that can possibly be accumulated in the reservoir would equal the difference between the reservoir-contents at those levels, as indicated by the shaded area in *Fig. 1.*

In the following Table are shown the maximum amounts of silt-deposit

Reservoir level, R.L: metres.	Water content: cubic metres × 10^6.	Maximum silt-deposit: cubic metres × 10^6.	Percentage reduction in reservoir capacity.
98	—	—	—
100	62	62	1·8
101	105	105	1·9
102	165	165	2·95
103	230	230	4·1
104	300	300	5·35
105	390	390	7·0

corresponding to given water-levels in the reservoir, together with the resulting percentage reduction in its capacity, which is equal to 5,600 cubic metres when filled to R.L. 122.

By "reservoir capacity" is meant the storage capacity of the reservoir above the natural river discharge. Owing to the steep slope of the river

in flood the reservoir content during the flood season is much smaller than its content during the remainder of the year. In the calculations the content of the reservoir at flood-time has been taken, since silting occurs only at flood-time, and not after the subsidence of flood; at least, that is assumed when filling the reservoir to its full capacity.

The Paper is accompanied by one sheet of drawings, from which the Figure in the text has been prepared.

OBITUARY.

SIR HERBERT NIGEL GRESLEY, C.B.E., was born at Seale, Leicestershire, on the 19th June, 1876, and died at Watton House, Hertford, on the 5th April, 1941. He was educated at Marlborough, and in 1893 became an apprentice on the London and North Western Railway. He then served for 2 years as a pupil on the Lancashire and Yorkshire Railway, and after a short period as test-room assistant was transferred to Blackpool, first as running-shed foreman and later as an outdoor assistant in the carriage and wagon department. In 1899 he became assistant works manager at the Newton Heath works of the Lancashire and Yorkshire Railway. In 1902 he was appointed manager, and in 1904 assistant superintendent. In 1905 he joined the Great Northern Railway as carriage and wagon superintendent at the Doncaster works, and 6 years later was appointed chief mechanical engineer. Following the railway re-grouping of 1922, he was appointed chief mechanical engineer of the London and North Eastern Railway, which position he retained until his death. His many notable advancements in locomotive design included the introduction of the Gresley gear; his 4–6–2 ("Pacific") express engines; his 2–8–2 ("Mikado") engines; and his streamlined locomotives, "Silver Jubilee" and "Coronation." For his achievements he received the degree of D.Sc. (*honoris causa*) from the University of Manchester in 1936.

During the war in 1914–1918 he served as a Lieutenant-Colonel in the Engineer and Railway Staff Corps, R.E.(T.). He was awarded the C.B.E. in 1920, and was knighted for his work as Chairman of the Board of Trade Committee on the steering gear of ships. He was also a member of Committees appointed by the Ministry of Transport on Railway Electrification and Automatic Train Control, and of a committee of the Department of Scientific and Industrial Research to consider a national station for testing locomotives. He was for many years a Governor of Queen Mary College (formerly the East London College).

Sir Nigel was elected a Member of The Institution on the 3rd March, 1941. In 1921 he presented a Paper[1], in collaboration with the late Sir Henry Fowler, M. Inst. C.E., entitled "Trials in Connexion with the Application of the Vacuum Brake for Long Freight Trains", for which he was awarded a Telford gold medal. He was President of The Institution of Locomotive Engineers in 1934, and of the Institution of Mechanical Engineers in 1936.

In 1901 he married Ethel Frances Fullagar, who died in 1929, and had two sons and two daughters.

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[1] Min. Proc. Inst. C.E., vol. 213 (Session 1921–2, Part 1), p. 223.

ABSTRACTS OF THE CURRENT TECHNICAL LITERATURE OF ENGINEERING AND APPLIED SCIENCE.

ENGINEERING CONSTRUCTION.

Description of a Water-Tunnel and Apparatus for the Investigation of Flow Problems. A. Fage and J. H. Preston (**J. R. Aer. Soc.*, 45, *pp. 124–140 ; April 1941*).—The Authors describe a water-tunnel of the closed-return type especially suitable for the observation of flow near the surface of a long streamline body of revolution at Reynolds' numbers up to $1{\cdot}3 \times 10^6$. The water passes from the inlet chamber through a fine-mesh honeycomb into a settling-chamber 32 inches long and 20 inches in diameter ; thence it is conducted through an entrance nozzle, contracting in diameter from 20 inches to 7 inches, into the observation chamber, which is 32 inches long and 7 inches in diameter. The general character of flow in the observation chamber is indicated by the behaviour of filament bands of white photographic ink. A fluid-motion microscope allows the movements of small particles in a bright beam of light to be observed and measured. A second fluid-motion microscope, with an interruption, allows the speed of a particle to be determined from the length of its track recorded for a known time of exposure on a photographic plate.

A Consideration of Pile Driving with Application of Pile Loading Formula. R. D. Chellis (**J. Boston Soc. Civ. Engrs.*, 28, *26 pp. ; Jan. 1941*). The Author presents data collected from widely-scattered sources, with the object of correlating them to enable any normal pile-driving problem to be solved quickly and easily with a degree of accuracy adequate for all practical purposes, and with uniformity of results. He summarizes the work of Hiley, Redtenbacher, and von Terzaghi in a comprehensive formula, with tabulations of the various weights and dimensions of parts of pile-drivers and piles and of the several coefficients required in the formula, so that all necessary data are available to enable an engineer, by entering these values in the formula, to produce a simple formula of the *Engineering News* type for use in the field. This simplified formula will contain only two coefficients, determined by and constant for the particular conditions of the job, such as size and type of hammer, length and material of pipe, type of driving head, and driving conditions ; it will apply only

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Notes.—An asterisk prefixed to a reference, thus **Proc. Amer. Soc. Civ. Engrs.* denotes that the article is illustrated.

The abbreviated titles of periodicals are those used in the "World List of Scientific Periodicals" (Oxford, 1934).

to the particular set of conditions existing on the job. Three Appendixes contain details of the application of the formula to determine penetration and to derive the field formula; a tabulation of driving conditions and formulas used for various projects; and a summary of comparative results of a driving test made on eleven piles.

Concrete Deterioration at Parker Dam. R. F. Blanks (**Engng. News-Rec.*, 126, *462–465; 27 Mar. 1941*).—Less than 2 years after completion of the Parker dam, on the Colorado river, widespread random cracks appeared on the surface of the concrete. This was the only one of fifty-four concrete dams constructed by the U.S. Bureau of Reclamation to exhibit such disintegration; the materials had been carefully selected and subjected to the usual physical and concrete performance tests. It was found, by drilling, that cracks of ⅛ inch opening at the surface extended into the mass a distance of only 6–8 inches. The Author, after investigation, attributes the cracking to adverse chemical reaction between the alkalis of the cement and siliceous minerals in the aggregate. Pending further research on alkali-aggregate reactions, the Bureau of Reclamation has imposed a 0·60-per cent. limit of alkali-content for cement used on important work.

Simplified Theory of the Self-Anchored Suspension Bridge. C. H. Gronquist (**Proc. Amer. Soc. Civ. Engrs.*, 67, *177–197; Feb. 1941*).—The Author presents a theory intended to enable the practising engineer readily to apply the formulas derived to the design of the self-anchored suspension bridge. The formulas, which are based upon the elastic theory, have been derived for a three-span symmetrical bridge with continuous girders. By dropping the continuity terms, the formulas for girders hinged at the towers are obtained. Constant girder moment of inertia within each span is assumed; the effect of this approximation, as well as that of neglect of suspender elongation, is considered. A method of correcting the value of the horizontal load in the cable and girder, as computed by the elastic theory, in order to take into account the movement of the ends of the girder produced by its vertical deflexion, is indicated.

Floating Falsework for Bridge Erection. (**Engng. News-Rec.*, 126, *320–322; 27 Feb. 1941.*)—The Ludlow Ferry highway bridge, which crosses the Potomac river about 50 miles below Washington, D.C., is 10,050 feet long between abutments. The main channel crossing consists of an 800-foot curved chord cantilever span, providing 140 feet clearance for shipping, and two 367-foot anchor spans; between the cantilever and the shore on the Virginia side are twelve truss deck spans and girder spans of the Wichert type, ranging from 117 feet to 350 feet, and sixty simple girder spans of about 61·5 feet each, whilst on the Maryland side are ten truss deck spans and girder spans of the Wichert type, ranging from 117 feet

to 350 feet. The cantilever was erected by balancing about the main piers in the usual manner, except that the anchor spans, when about half completed, were joined to the anchor piers by temporary steelwork to reduce balancing and wind stresses. All but the last of the Wichert truss spans were erected over a temporary truss support, which was floated in and set on the piers; the last span was floated in and lowered into position without the aid of falsework. Girder spans over water were lifted into place by floating derricks, whilst those over land were set by travelling cranes. The bridge provides a roadway width of 24 feet between curbs, with an emergency footpath $1\frac{1}{2}$ foot wide on each side.

The Design of Beams in Steel-Frame Buildings. S. D. Lash (**Engng. J.*, 24, *188–195; Apr. 1941*).—The Author describes methods for design in which allowance is made for partial or complete restraint of the ends of the beams. For secondary beams it is assumed that a high degree of restraint is present, and it is demonstrated that this may be ensured by the use of a comparatively small quantity of special steel in the form of reinforcing-bars in the concrete floor-slab, or by plates attached to the top flanges of the beams. For beams connected to columns, curves are derived by means of which allowance may be made for the restraint produced, by simple connexions using clip angles. The design method is based upon the work of the British Steel Structures Research Committee, but is considered to be simpler to use and more accurate than the method recommended by that Committee. The Author gives a number of examples illustrating the application of the proposals, and the possible saving in weight of steel.

Waterval Boven Water-Supply, Transvaal. A. F. Bruyns Haylett (**Proc. S. Afr. Soc. Civ. Engrs., 1941, 8–27; Jan. 1941*).—Waterval Boven, a railway depot on the Eastern Transvaal section of the South African Railways, is situated at an altitude of 4,826 feet above sea-level. The water-supply is derived from the Elands river, and especially during the summer, was very unsatisfactory, the water resembling pea soup. The Author describes the arrangements by which a chlorinated filtered supply was obtained. The scheme consisted of a 750,000-gallon reservoir of treated river water, a total storage of 150,000 gallons of drinking water, a filter, and a chlorination plant. The principal feature is the sponge filter, which is described in detail. Rated at 100,000 gallons for 24 hours, it consists of a cast-iron cylinder 2 feet in diameter, and about 9 feet long, divided into six cells, each of which is filled with about 110 unbleached sea sponges. The Author states that the rate of filtration is sixteen times as high as that of any make of rapid sand filter, whilst cleaning is easy and economical. He also describes the mixing-plant, the pre-settling ponds, the main reservoir, and the 6-inch gravitation main from the river. In an appendix the results of tests made on a sponge filter in a Johannesburg mine are given in detail.

Creep of Rails. S. N. Dutt (**Indian & Eastern Engr.*, 87, *196–199 ; Aug. 1940*).—The Author enumerates the destructive effects of creep upon railway track, discusses its causes, direction, magnitude, and extent, summarizes the cost of pulling back rails, and describes measures for its prevention, with their various merits and demerits. He describes the Simpless creep-arrester, devised by himself, and states that it has successfully prevented rail creep up to 6 feet 8 inches in actual practice. The device consists of a metal frame formed of two angle-iron members connected together by subsidiary members which cross diagonally. The length of the two parallel members is such as to keep the two opposite rails at the correct gauge-distance apart. The arrester may be fitted either at the rail-joint or at the middle of a rail ; the Author prefers the latter.

Supporting Strengths of Cast-Iron Pipe for Water and Gas Service. W. J. Schlick (**Bull. Iowa Engng. Expt. Stn., No. 146, 121 pp. ; 1940*). The Author presents experimental results which, in conjunction with load calculations developed earlier at the Station, provide a convenient and reliable means for the structural design of any line of cast-iron pipe subjected to both external load and internal pressure. The tests were made on 6-inch, 12-inch, 20-inch, and 48-inch pit-cast pipe under a range of representative loading conditions and internal pressures. The test procedure is described in detail, and the results obtained are recorded in numerous Tables and curves. A bibliography of the subject is appended to the Report.

Corrosion Experience at Richardton, North Dakota. L. K. Clark (**J. Amer. Waterw. Ass.*, 33, *293–298 ; Feb. 1941*).—The water-supply is derived from a well 50 feet deep, from which two tunnels extend horizontally for 60 feet. In February 1939 water samples indicated an iron-content of about 9 pails per million. Towels became brick-red after one day's use ; it was impossible to wash clothes white ; and the hot water, in addition to its high iron content, had a very disagreeable odour. It was necessary to flush the city hydrants every day. Chemical feeders were therefore installed, one to feed soda ash to raise the *pH*-value and the other to add a small quantity of a commercial phosphate stabilizer to prevent precipitation of calcium following the rise in *pH* ; about 4 lb. of soda ash per 1,000 gallons was needed to raise the *pH*-value to a point where corrosion would cease. The Author concludes that considerable corrosion may be expected from the high free CO_2-content of well waters, and that CO_2 does not dissipate in appreciable quantities from the surface of the water in a well, even 16 hours after pumping. He states that with the raising of the *pH*-value consumption of water decreased considerably, as it was no longer necessary to run the water to obtain a clear supply.

MECHANICAL ENGINEERING.

Gas-Fired Steam Boilers for the Disposal of Low-Grade Fuel. James Frame (**Min. Elect. Engr.*, 21, *231–236; Mar. 1941*). The Author describes a plant operated by the Lothian Coal Company, Ltd., utilizing gas-producers and the gas-firing of steam boilers, which overcomes the ash problem and permits the use of a fuel containing only 50 per cent. of combustible material. A battery of five Kerpely gas-producers is installed at the rear of its boilers, which are of the enclosed type, mechanically fed, with revolving grate and bottom, each capable of generating gas to evaporate 10,000 lb. of water per hour when gasifying a mixture of 50 per cent. combustible and 50 per cent. ash. The operation of the plant is described in detail and performance data are tabulated. There are nine boilers, eight of which have to do the work, leaving one off for cleaning and repairs. The duty of each boiler is to evaporate 6,250 lb. of water per hour, and it is confidently expected that this will be maintained.

Power-Station Auxiliaries. T. H. Carr (**Electrician*, 126, *101–102; 14 Feb. 1941*).—The problem of the design and arrangement of auxiliary plant is rather complicated, and the solution must always represent a compromise between conflicting factors. The station auxiliary plant load may amount to anything from 4 to 10 per cent. of the station load. The Author suggests various arrangements and discusses their respective merits. He classifies auxiliaries as those which must be kept in almost continuous service if the station is to maintain its output, and those which can be taken out of service for appreciable periods; and he emphasizes the fact that, so far as the supply of electricity is concerned, an essential auxiliary is just as important as a main unit, whilst a main unit breakdown is probably more excusable in the view of a consumer.

New General Utility Locomotives for the London and North Eastern Railway. (**Rly. Gazette*, 74, *221–223; 28 Feb. 1941.*)—The new locomotives, of the 2–6–2 type, classified as "V4", are of similar design to the "Green Arrow" (V2) class, but are lighter and are suitable for working over sections of the line where the 22-ton axle load of the latter is prohibited. They are of medium power and incorporate high boiler-pressure, high superheat, long valve-travel, and the Gresley gear. Weight has been saved by the use of 2-per cent. nickel-steel for the boiler barrel and by extensive use of fabricated construction instead of steel castings for the dragbox, main frame stays, and boiler supports. The engine in working order weighs about 70 tons, in comparison with 93 tons for the "V2" class. The three cylinders are 15 inches diameter by 26 inches stroke. All of the revolving and 40 per cent. of the reciprocating masses are balanced. The principal particulars are as follows: wheels, coupled, 5 feet 8 inches diameter; boiler, outside length of firebox overall, 9 feet $1\frac{5}{8}$ inches;

working pressure 250 lb. per square inch; grate-area, 28·5 square feet; total evaporative heating surface 1,444·1 square feet; tractive force (at 85 per cent. boiler-pressure) 27,420 lb.; total adhesion weight 48 tons 1 hundredweight; tender capacity, 6 tons of coal and 3,500 gallons of water.

New Goods Locomotives for the Western Maryland Railway. (**Rly. Age, Chicago*, 110, *209–215; 25 Jan. 1941.*)—Twelve 4–6–6–4 single-expansion articulated locomotives are being delivered by the Baldwin Locomotive Works, for operation between Hagerstown, Md., and Connellsville, Pa., a distance of 171½ miles, including severe gradients. These locomotives have driving-wheels 69 inches in diameter. The boiler is of the straight-top type, with three barrel courses. The grate-area is 118·8 square feet, the combined heating surface is 5,570 square feet, and the working pressure is 250 lb. per square inch. The four cylinders are 22 inches diameter by 32 inches stroke. The firebox and combustion-chamber are of welded construction; Duplex thermic siphons are fitted in the firebox. The weight on the driving-wheels is 402,266 lb. and the total engine weight is 601,000 lb. The tender, of the water-bottom type, has a capacity of 30 tons of fuel and 22,000 (U.S.) gallons of water.

New 2,000-Horsepower Diesel-Electric Locomotives for the Chicago, Rock Island and Pacific Railway. (**Rly. Mech. Eng.*, 115, *89–92; Mar. 1941.*)—Two locomotives have been delivered for service on the "Arizona Limited" passenger train, which runs between Chicago and Tucumcari, Arizona. Each is equipped with two six-cylinder vertical four-stroke-cycle turbo-charged diesel engines, each developing 1,000 horse-power at a normal running speed of 740 revolutions per minute, and driving a direct-connected main generator, an auxiliary generator mounted on an extension of the shaft of the main generator, and a split-pole exciter mounted on the top of the auxiliary generator. The locomotives are designed for a maximum speed of 120 miles per hour. Their length overall is 74 feet 9¼ inches, maximum width 10 feet 6 inches, and maximum height 14 feet 4 inches. The weight on the driving-wheels (four pairs), which are 40 inches in diameter, is 220,000 lb., and on the idling wheels (two pairs) also 40 inches in diameter), 110,000 lb. The starting tractive force, at 24-per cent. adhesion, is 53,000 lb.

A New Electric Locomotive for the London and North Eastern Railway. (**Rly. Gazette*, 74, *248–249; 7 Mar. 1941*).—Although work on the Manchester–Sheffield electrification scheme was largely suspended on the outbreak of war, a decision was made to proceed with one locomotive, so that it could be thoroughly tested on a short section of line equipped with a 1,500-volt direct-current catenary system. The locomotive is designed to haul both passenger and main line goods traffic on long gradients of

1 in 100, and to develop a maximum speed of 65 miles per hour; the axle arrangement is $B_0 + B_0$, and the maximum axle load is $22\frac{1}{2}$ tons. The steel body and girder underframe are carried on two four-wheel bogies, on which are mounted the buffing and drawgear. The wheels are 50 inches in diameter, and the wheelbase is 11 feet 6 inches. Westinghouse brakes, a hand brake, and vacuum apparatus to operate the brakes on passenger and goods trains are fitted. A driving compartment is provided at each end. Current is collected from the overhead line by two pantographs of light but rigid design. The control gear is of the electro-pneumatic switch type; remote control of the electro-pneumatic contactors and drum-type switches for reversing, etc., is effected by 50-volt circuits from the master controller. Four force-ventilated nose-suspended traction motors are installed, the two on each bogie being connected permanently in series, with 750 volts across the brushes. They can be operated in series and parallel combinations, and four stages of field weakening are provided. A maximum 1-hour rating of 467 horse-power per motor is obtainable, but the normal 1-hour value at full field is 435 horse-power per motor at 26 miles per hour and with a tractive effort of 6,250 lb. per motor. The total maximum starting tractive effort is 45,000 lb., giving the ample adhesion factor of 4·36.

MINING ENGINEERING.

The Deviation and Survey of a Horizontal Borehole. G. A. CORDEN (**Trans. Instn. Min. Engrs.*, 100, *153–158; Feb. 1941*).—The Author describes a simple method for making an approximate survey of a horizontal borehole, the advantages of which are enumerated as follows: (1) the position of the in-bye end of a borehole can be plotted and the intersection or otherwise of boreholes proved beyond doubt, thus providing a greater measure of protection for all mining operations within a bored area; (2) as the actual position of the holes can be plotted, less allowance for possible deviation need be made when advancing headings within the protected area; (3) deviation of the hole can be detected at an early stage, and if excessive the hole can be abandoned and a new one commenced. After consideration of several methods, it was decided to make an instrument in which a small compass-needle suspended in a gelatine solution is used, which can be placed in a borehole to obtain magnetic bearings at various points along the hole. The instrument is described in detail, and the corrected bearings obtained at points along the borehole are tabulated. A heading driven later met the borehole within $2\frac{1}{2}$ feet of its surveyed position.

The Stability of Supports used Underground. J. W. BOWEN (**J. S. Afr. Instn. Engrs.*, 39, *157–176; Feb. 1941*).—The Author reviews briefly the development of various types of supports used underground in South

Africa and gives detailed descriptions of tests carried out at the Government Mechanical Laboratory at Cottesloe. All packs tested were built 2 feet square and 2 feet high, and were composed of timber having a moisture content of +30 per cent.; concrete disks, pigsties, and Sloan-wire packs were also tested, and in each case one pack, with the exception of the Sloan-wire pack, was tested as a 2-foot high support, so that direct-comparison could be made with the results obtained with the timber packs. The rate of loading of the testing-machine was constant throughout. The object was to obtain an idea of the load actually supported by various types of mats, concrete disks, and stone-filled pigsties. The results are presented in Tables and curves, and conclusions, based upon load compression curves from various types of support tested to a maximum load of 1,000 tons, are drawn for (1) wedges; (2) timber poles; (3) concrete disks; (4) chock mats; (5) pigsties; (6) Sloan-wire packs. The Author states that as poles exceeding 9 inches in diameter withstand greater loads than do concrete disks 24 inches in diameter, and as they prove equally satisfactory for most purposes, there appears to be no reason why timber cannot be used instead of concrete.

The Strength of Undermined Strata. W. H. Evans (**Bull. Inst. Min. & Metallurgy, No. 349, 26 pp.; April 1941*).—The Author proposes a "voussoir-beam" theory to account for the relative stability of roof strata that are too cracked transversely to act as simple beams, and develops formulas for the moment of resistance due to horizontal thrust and end-fixing moments, taking into account the effect of deflexion in reducing the effective height of the arch. Curves have been plotted showing the maximum spans for self-supporting voussoir-beams, of various physical properties, for given thicknesses. The effect of the yield of the abutments is also taken into consideration. The Author describes experiments on voussoir-beams composed of ordinary building bricks. The results of these tests confirm that, provided the end-reactions are adequate, a voussoir-beam can be stable under the load of its own weight even when it is traversed by numerous breaks and is incapable of taking tensile stress.

A Critical Review of Dust Sampling Methods employed in Witwatersrand Gold Mines. J. de V. Lambrechts (**J. Chem. Met. Min. Soc. S. Afr.*, 41, *209–240; Dec. 1940*).—The Author makes a critical comparison of the konimeter and the thermal precipitator, which are the instruments most generally used on the Witwatersrand for dust-sampling and discusses their respective advantages and limitations, in the light of experiments which are described in detail. The dust-counts obtained from the two instruments are summarized in a Table, and the konimeter efficiencies are expressed as percentages relative to the thermal precipitator counts. In his conclusions the Author states that, generally speaking, the konimeter, with the recommended new method of treatment, should give

an entirely reliable indication of the dust risk at the time and place of sampling; the thermal precipitator is invaluable for special investigations, because, in addition to giving a continuous sample, it furnishes information in regard to both numbers and sizes of dust particles. A bibliography of the subject is appended to the Paper.

Permissible Dust Count Proposed for Utah Mines. (*Mining World* (*U.S.A.*), 3, *23; Feb. 1941.*)—In a report on a 2-years' study of occupational disease hazards conducted in Utah it is concluded that proper application of control measures can effectively curb silicosis and considerably reduce pulmonary tuberculosis among non-ferrous mine and smelter workers. The main recommendation made in the report is the establishment of a "tentative maximum permissible limit" of dust-concentration at from 5 to 10 million particles per cubic foot of air for metalliferous mines, and 20 million particles per cubic foot for coal mines. The recommendations of the Utah Slate Board of Health regarding "tentative maximum permissible limits" are concerned with dust-particles of all types, and not specifically with counts of particles of free silica. The average content of free silica in samples taken from metalliferous mine airways was 22·6 per cent. of the total number of particles.

Moisture Relations of Banded Ingredients in an Illinois Coal. O. W. Rees, G. W. Land, and F. H. Reed (*Ind. Eng. Chem.*, 33, *416–419; March 1941*).—The moisture : humidity relations of the banded ingredients vary in relation to each other and to the whole coal. The moisture : humidity curve for vitrain lies above that of whole coal, that of clarain very near that of whole coal, and fusain below all others up to 96 per cent. humidity, at which value it rises sharply well above all others. The moisture-values of the whole coal at different humidities appear to be weighted composites of the moisture-values of the component ingredients. The equilibration and calculated pore-size data appear to correlate well with the capillary theory for the occurrence of moisture in coal. The variation in the moisture-relations of whole coal and its component banded ingredients, as shown by studies on this one coal, have a distinct bearing upon moisture in commercially-prepared coals.

Thermodynamic Treatment of the Swelling Pressure of Coal. W. Fuchs (*J. Franklin Inst.*, 231, *103–119; Feb. 1941*).—For the past few years the swelling pressure of coal during carbonization has been under investigation at the Pennsylvania State College School of Mineral Industries, but no results have yet been published. The Author approaches the problem along theoretical lines, and deals with fundamental concepts pertaining to the subject, the influence of time, and the theory of measuring devices. He derives formulas for the swelling pressure from the first and second laws of thermodynamics, and makes estimations for various

otherwise unavailable thermodynamic properties. The application of the formulas is illustrated by examples.

Investigation of the Spontaneous Ignition of Coal from the Petrographic Standpoint. E. NÖTZOLD (**Glückauf*, 76, *381–388; 393–397; 1940*).—The ease with which the coal of the Heinrich Robert mine ignites spontaneously has caused this coal to be studied. The Author advances the theory that the process of self-oxidation is a result of the reforming of the humus compounds which were made unstable by the loss of gases under pressure. Certain vitrains are most susceptible to spontaneous combustion owing to the liberation of CH_4 and the subsequent addition of O_2 to stabilize the compound, with simultaneous rise in temperature. Pressure set up by earth movements is responsible for the change in the characteristics of the vitrains, whilst the other coal components do not appear to be affected.

Firedamp Ignition by Compressed-Air Discharges. I. C. F. STATHAM (**Iron Coal Tr. Rev.*, 142, *315–316; 345–346; 14 and 21 Mar. 1941*).—The Author reviews investigations of the problem of ignition of firedamp by electrostatic discharges, carried out in Great Britain and on the Continent, and describes briefly the circumstances attending an ignition of firedamp at a ripping lip in a South Yorkshire colliery in November 1939, investigation of which suggested that the ignition resulted from sparking between the electrostatically-charged coupling at the end of the hose and the earthed girder around which it was coiled. Experiments were therefore made at the University of Sheffield on the length of hose and the coupling to study the possibility of ignition in that manner. The experiments were carried out (1) with the coupling suspended in air; (2) with the coupling clamped near to an earthed plate; (3) in an explosion chamber; (4) with an air-filter at the end of the hose. The Author concludes that the danger of electrostatic discharges is not present in all mines in equal degree, and is likely to arise only under certain hygrometrical conditions. In wet pits with moist atmospheres no risk is to be apprehended; but the danger is likely to arise when the relative humidity is below about 70 per cent.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.

NOTE.—*Pages* [1] *to* [18] *can be omitted when the Journal is bound in volume form.*

NOTICES

No. 7, 1940—41

JUNE, 1941

MEETINGS, SESSION 1940–41.

ANNUAL GENERAL MEETING.

It has not been found possible to prepare the Report of the Annual General Meeting, which was held on the 27th May, in time for inclusion in this Number of the Journal. It will, however, be published later on.

SPECIAL ANNOUNCEMENTS.

COURSES OF INSTRUCTION IN ENGINEERING ECONOMICS AND MANAGEMENT AND THE AESTHETICS OF CONSTRUCTIONAL DESIGN AT CAMBRIDGE UNIVERSITY.

The Council of The Institution have recently given consideration to the need to foster amongst engineers the closer study of :—

(*a*) the economics of engineering projects,
(*b*) the organization and management of engineering work, and
(*c*) the relations of aesthetic considerations to engineering design and construction.

In the past these three subjects have not generally been included in an engineer's education and training, but have been left to be acquired by practical experience spread over many years. Together they include many aspects of engineering work which require to be kept continuously under review in the light of changing economic conditions and the incidence of scientific research on the use of materials and on design.

The Council believe that in the period of reconstruction and development which will follow the present war these subjects will become of increasing importance, and in the national interest should form a part of the equipment with which engineers should be furnished in order that they may adequately play their part in the task before them.

The Council accordingly intend to examine all possible means by which The Institution can influence the better understanding of engineering economics and aesthetics, not only by students resident at the Universities but also by those otherwise engaged in their engineering training, so that eventually a study of these subjects may form an integral part in the recognized education of all engineers.

The Council have approached the Vice-Chancellor of Cambridge University with an offer to finance for a period of five years a lectureship on the subjects envisaged in the hope that they would in due course form part of the engineering curriculum of the Mechanical Sciences Tripos.

The proposal was cordially welcomed and has now been accepted in principle by the Council of the Senate of the University, but its full adoption will necessitate some reconsideration of the scheme of instruction in the Engineering School and possibly of the Mechanical Sciences Tripos, which it would be impracticable to undertake in war time. It is proposed, however, that a beginning should be made in the next academical year, commencing in the autumn by inviting a number of eminent engineers and others to visit the University to give either single lectures or short courses on subjects coming within the terms of the proposal.

SERVICE IN THE ARMED FORCES.

Corporate Members who are free and who desire to offer themselves for service in the Armed Forces may notify the Secretary, as vacancies in various technical branches of the Services are intimated to The Institution from time to time, during intervals of publication of each Number of the Journal.

RECORD OF SERVICE IN THE FORCES.

For office purposes, a record is being kept of members' service with H.M. Forces, and Corporate Members and Students who have not already done so are asked to inform the Secretary of such service, i.e. unit, rank, promotions, decorations, etc. Further, practical use is made of such information when inquiries from the Services are received by The Institution.

ARMY OFFICERS' EMERGENCY RESERVE.

The attention of corporate members of The Institution is directed, at the request of the War Office, to the following statement (reprinted from *The Times*) :—

> " While the main source of supply for the Army has been the Officer Cadet Training Units, which are themselves filled by men selected from the ranks, a very large number of commissions have been granted through the Army Officers' Emergency Reserve, mainly to those who have already served in the last war, or to those required

in appointments in which some special technical qualifications are needed rather than military experience or training.

Last year a comprehensive organization for interviewing candidates was set up. Candidates for technical arms are interviewed by Selected Officers of those arms while those without special technical qualifications are seen by Interviewing Boards which have been established in London and many provincial centres. A notable addition to these Boards has been the inclusion of a civil member of wide industrial or commercial experience. The advice and assistance which these gentlemen have been able to give has been of the greatest value.

It is often forgotten that the Army Officers' Emergency Reserve is intended to be a reserve from which gentlemen can be called up for commissioning as they are required. It is not, and was never intended to be, an employment bureau which can undertake to find employment for applicants.

Registration in the A.O.E.R. carries no guarantee of employment and no candidate should give up any other form of employment merely because he has been accepted for registration. Whether any member of the Reserve is called up and when, must depend upon his qualifications and medical category and on the varying demands for the different classes of Officer. In some cases there is a large reserve registered; in others, more particularly for Royal Engineers, Royal Army Service Corps, and Royal Army Ordnance Corps, demand is large and continuous and suitable candidates in certain classes for these Corps are usually called up soon after registration. The Pioneer Corps also makes considerable demands to meet its growing needs.

The A.O.E.R. is still open to receive applications from those who have not yet felt able to offer their services. For obvious reasons the number of vacancies in each Corps cannot be stated, but candidates accepted for registration for a technical arm would probably be called up for service in no long time after registering."

TRANSFER TO THE CORPS OF ROYAL ENGINEERS.

It is understood that Corporate Members and Students of The Institution who are serving in the Army, but not in the Royal Engineers, and who desire to be transferred to that Corps (with a view to consideration of their qualifications for Commissions) should apply to their respective Commanding Officers for permission to so transfer. It is also understood that, in suitable cases, such applications will be acceded to.

The Secretary will be pleased to furnish certificates of membership of The Institution, upon request, for attachment to such applications.

GENERAL ANNOUNCEMENTS.

THE JOURNAL.

The next Number of the Journal will be published on the 15th October.

AWARDS FOR PAPERS.

The special thanks of The Institution have been given to Mr. A. G. Vaughan-Lee, Author of a Paper on "The Mohammad Aly Barrages, Egypt", and to Mr. H. C. Whitehead, Author of a Paper on "The Design of Sewage-Purification Works", who, as Members of Council, were ineligible to receive Awards.

The following Awards have been made for Papers read and discussed in Session 1940–41 :—

The Baker Gold Medal to :—

Oscar Faber, O.B.E., D.C.L., D.Sc., M. Inst. C.E., for his Paper on "Aesthetics of Engineering Structures."

A Telford Premium to :—

A. J. H. Clayton, B.Sc. (Eng.), Assoc. M. Inst. C.E., for his Paper on "Road Traffic Calculations."

The Awards for other Papers published with and without written discussion will appear in the October Number of the Journal.

EXAMINATIONS.

The Common Preliminary Examination, which has now taken the place of The Institution Preliminary Examination, is to be held in London and the Provinces on the 7th to the 10th October, inclusive.

The Associate Membership Examination is, in view of the war conditions, to be held in London and the Provinces during the week commencing the 12th October.

Completed applications to attend these Examinations should be placed in the Secretary's hands by the 31st August.

It would be of convenience if Students of The Institution entering for the Associate Membership Examination would forward their applications a fortnight before that date.

TRANSFERS, ELECTIONS AND ADMISSIONS.

Since the 18th March, 1941, the following elections have taken place :—

Meeting.	*Member.*	*Associate Members.*
29 April, 1941.	—	41
27 May, 1941.	1	22

and during the same period the Council have transferred six Associate Members to the class of Members, and have admitted fifty-six Students.

DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations :—

Deaths.

Brodie, William. (E. 1899. T. 1914.)	*Member.*
Connell, Cecil Bourke. (E. 1912. T. 1930.)	,,
Cook, Ernest Walter. (E. 1905. 1937.)	,,
Deverell, Thomas Clark. (E. 1886. T. 1900).	,,
Garvie, Robert Halley, B.Sc. (E. 1902. T. 1916.)	,,
Gresley, *Sir* Herbert Nigel, C.B.E. (E. 1914.)	,,
Hunt, John Theodore. (E. 1905. T. 1912.)	,,
Hunt, Philip Charles Holmes. (E. 1911.)	,,
Newell, Hugh Hamilton, C.B.E. (E. 1929)	,,
Warren, John Russell, M.C., B.Sc. (E. 1915. T. 1926.)	,,
*Ball, Lawrence Joseph. (E. 1939.)	*Associate Member.*
Birt, Raymond John. (E. 1890.)	,, ,,
Brown, Fergus. (E. 1925.)	,, ,,
Cobb, Frederic Edward Theodore. (E. 1885.)	,, ,,
*Coombs, Herbert Martin, B.Sc. (E. 1938.)	,, ,,
Cripps, Frederick Southwell. (E. 1888.)	,, ,,
*Gray, William John Alexander. (E. 1935.)	,, ,,
*Hayward, William Edwin. (E. 1937.)	,, ,,
Lembcke, Gustave Michael de, M.A. (E. 1921.)	,, ,,
Murray, Charles. (E. 1890.)	,, ,,
Rowe, John Bedford. (E. 1905.)	,, ,,
Stuart, Martin Richard Furnneaux Henessy. (E. 1920.)	,, ,,
Tippett, David Gibb, B.Sc. (E. 1933.)	,, ,,
Chapman, Donald Mackenzie. (A. 1939.)	*Student.*
*Hire, Merrik Hubert Eric. (A. 1937.)	,,
Lyons, John Denys Marsh, B.A. (A. 1936.)	,,
*Vaughan, William David. (A. 1933.)	,,
*Wadsworth, Frank Thomas Bernard. (A. 1938.)	,,

* *Killed on active service.*

Resignations.

Basta, Habib. (E. 1907.)	*Associate Member.*
Bean, Benjamin Donald Hewitt. (E. 1913.)	,, ,,
Beaumont, Geoffrey James Le Breton. (E. 1931.)	,, ,,
Cassidy, George Leonard. (E. 1929.)	,, ,,
Chute, John Henry Chaloner. (E. 1893.)	,, ,,
Crabtree, Walter Robinson, M.Sc. (E. 1895.)	,, ,,
Edmunds, Howard Maurice, M.C. (E. 1910.)	,, ,,

Lawn, James Gunson, C.B.E. (E. 1896.)	Associate Member.
Manson, Frederick Percy. (E. 1907.)	,, ,,
Mutimer, Geoffrey Horace. (E. 1932.)	,, ,,
Nash, Henry Algernon Fraser. (E. 1920.)	,, ,,
Andrews, Edwin Alexander. (A. 1937.)	Student.
Bruce, William George. (A. 1932.)	,,
McFarlane, John Falconer. (A. 1939.)	,,

RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

Acoustics. Olsen, H. F. "Elements of Acoustical Engineering." 1940. Chapman & Hall. 30*s.*

Aircraft. Chatfield, C. H., and others. "The Airplane and its Engine." 4th ed. 1940. McGraw-Hill. 21*s.*

Automobiles. Anonymous. "Autocar Handbook." 14th ed. 1941. Iliffe. 3*s.*

Ballistics. Burrard, G. "Identification of Firearms and Forensic Ballistics." 1934. Jenkins. 12*s.* 6*d.*

Bentonite. Davis, C. W., and Vacher, H. C. "Bentonite: Properties, Mining, Preparation, and Utilization." (U.S. Bur. Mines Technical Paper 609.) 1940. Supt. of Documents, Washington. 15 cents.

Building. Schobinger, G., and Lackey, A. M. "Business Methods in the Building Field." 1940. McGraw-Hill. 24*s.* 6*d.*

Camouflage. Chesney, C. H. R. "The Art of Camouflage." 1941. Hale. 8*s.* 6*d.*

Climate. Kendrew, W. G. "Climate." 2nd ed. 1938. Clarendon Press. 15*s.*

Concrete. Urquhart, L. C., and O'Rourke, C. E. "Design of Concrete Structures." 4th ed. 1940. McGraw-Hill. 31*s.* 6*d.*

*Dams and Reservoirs. U.S. War Dept. Corps of Engineers, U.S. Army. "Report on the Slide of a Portion of the Upstream Face of the Fort Peck Dam, Fort Peck, Montana." 1939. Washington. Supt. of Documents. No price.

Deserts. Pickwell, G. "Deserts." 1939. McGraw-Hill. 17*s.* 6*d.*

*Dictionaries. American Association of Port Authorities. "A Port Dictionary of Technical Terms." 1940. The Association, 2223 Short Street, New Orleans. 1 dollar.

Education. Baker, H. "Technical Education after the War." 1941. Paper of the Association of Technical Institutions. No price.

Estimating. *See* Ratefixing.

Factories. Emsley, H. H., and Loxham, J. "Factory Costing and Organisation." 2nd ed. 1939. Constable. 9*s.*

Firearms. *See* Ballistics.

Industrial Welfare. "Proceedings of Industrial Safety Conference, November 8, 1940." Bulletin No. 38. Virginia Polytechnic Institute, Blacksburg. No price.

Irrigation. Comisión Nacional de Irrigación. "La Obra de la Comisión Nacional de Irrigación . . . 2 vols. Mexico, 1940. No price.

Laboratories. Strong, J., and others. "Modern Physical Laboratory Practice." 1939. Blackie. 25*s.*

*Libraries. Thornton, J. L. "Special Library Methods." 1940. Grafton. 10*s.* 6*d.*

MACHINE TOOLS. TOWN, H. C. "Modern Machine Tools." 1940. Pitman. 30*s.*

MAGNESIUM. BECK, A. "Technology of Magnesium and its Alloys." 1940. F. A. Hughes. 30*s.*

MATERIALS—TESTING. BEAUMONT, R. A. "Mechanical Testing of Metallic Materials." 1940. Pitman. 6*s.*

—— SKERRY, E. "Testing of Metallic Materials." 1938. Bunhill Publications. 3*s.* 6*d.*

MATHEMATICS. BELL, E. T. "The Development of Mathematics." 1940. McGraw-Hill. 31*s.* 6*d.*

—— PLUMMER, H. C. "4-Figure Tables with Mathematical Formulæ." 1941. Macmillan. 3*s.* 6*d.*

—— UNDERWOOD, R. S., and SPARKS, F. W. "Living Mathematics." 1940. McGraw-Hill. 21*s.*

See also STATISTICS.

MICROSCOPY. *See* MINERALS.

MINERALS. SMITH, H. G. "Minerals and the Microscope." 4th ed. 1940. Murby. 5*s.* 6*d.*

MUNICIPALITIES. FINER, H. "Municipal Trading." 1941. Allen and Unwin. 16*s.*

PETROLOGY. HATCH, F. H., and WILLS, A. K. "Petrology of the Igneous Rocks." 9th ed. 1937. Allen and Unwin. 15*s.*

—— HATCH, F. H., and RASTALL, R. H. "Petrology of the Sedimentary Rocks." 3rd ed. 1938. Allen and Unwin. 15*s.*

PHYSICS. *See* LABORATORIES.

PIPES AND TUBES. SCHLICK, W. J. "Supporting Strength of Cast Iron Pipes for Water and Gas Service." Bulletin 146. 1940. Iowa Eng. Exp. Stn., Ames. Gratis.

PLASTICS. "Plastes." "Plastics in Industry." 1940. Chapman & Hall. 12*s.* 6*d.*

PORTS. *See* DICTIONARIES.

POWER FACTOR ECONOMICS. ROGERS, P. L. "Power Factor Economics." 1939. Chapman & Hall. 15*s.*

PUBLIC SERVICES. WICKWAR, W. H. "The Public Services. A Historical Survey." 1938. Cobden-Sanderson. 10*s.* 6*d.*

PUMPS. KRISTAL, F. A., and ANNETT, F. A. "Pumps: Types, Selection, Installation, Operation, and Maintenance." 1940. McGraw-Hill. 24*s.*

RADIATORS. RUBERT, K. F. "Heat Emission from Radiators." 1937. Bulletin 24, Cornell Eng. Exp. Stn. Price 45 cents.

RADIO ENGINEERING. RAPSON, E. T. A. "Experimental Radio Engineering." 1940. Pitman. 8*s.* 6*d.*

RATEFIXING. FREEMAN, J. C. "Ratefixing and Estimating for Engineers." 1941. Pitman. 7*s.* 6*d.*

SAFETY MEASURES. *See* INDUSTRIAL WELFARE.

*SCIENCE. HALDANE, J. B. S. "Science in Peace and War." 1941. Lawrence and Wishart. 5*s.*

SEWAGE DISPOSAL AND SEWERAGE. KEEFER, C. E. "Sewage Treatment Works. Administration and Operation." 1940. McGraw-Hill. 42*s.*

STATISTICS. PETERS, C., and VAN VOORHIS, W. R. "Statistical Procedures and their Mathematical Bases." 1940. McGraw-Hill. 31*s.* 6*d.*

—— YULE, G. U., and KENDALL, M. G. "An Introduction to the Theory of Statistics." 1940. Griffin. 24*s.*

STRUCTURES. TIMOSHENKO, S. "Theory of Plates and Shells." 1940. McGraw-Hill. 42*s*.

*TOWN AND AREA PLANNING. WEST MIDDLESEX JOINT TOWN PLANNING COMMITTEE. "West Middlesex Regional Planning Scheme. Report." 1924.

TUNNELLING AND TUNNELS. RICHARDSON, H. W., and MAYO, R. S. "Practical Tunnel Driving." 1941. McGraw-Hill. 35*s*.

WAR. FALLS, C. "The Nature of Modern Warfare." 1941. Methuen. 4*s*.

—— MURRAY, S. "War Damage Act, 1941." 1941. Eyre & Spottiswoode. 6*s*. 6*d*.

—— PEMBERTON-BILLING, N. "Defence against the Night Bomber." 1940. Hale. 2*s*. 6*d*.

WORKS ORGANIZATION. LARKIN, E. J. "Works Organization and Management." 1940. Pitman. 35*s*.

(* *The foregoing books, with the exception of those marked with an asterisk, may be borrowed from the Loan Library.*)

INSTITUTION LUNCHEON.

A Luncheon was held by The Institution at Grosvenor House, Park Lane, on Wednesday, 30 April, when 227 members and guests were present. Sir Leopold Savile, K.C.B., President, was in the Chair.

The President, after proposing the toast of "The King", which was loyally pledged, said: What I am about to say now is not on the official programme, but I have been authorized by the Civil Lord of the Admiralty to state that the Prime Minister announced to-day in the House of Commons that of about 60,000 of our troops in Greece, over 45,000 have been evacuated. (Applause.) I ask you—this also is not on the programme—to charge your glasses and drink to the health of the Prime Minister.

Lord REITH OF STONEHAVEN, P.C., G.C.V.O., G.B.E., D.C.L., LL.D., M. Inst. C.E., Minister of Works and Buildings, then proposed the Toast of "The Institution of Civil Engineers." He said: Last year I addressed you unexpectedly and briefly from the left of the Chair. I told you some harrowing tales of early struggles with such of your great ones as Sir Ernest Moir and Sir Alexander Gibb. To-day I might tell of struggles with such of their successors as Mr. Hugh Beaver and Colonel Howard Humphreys, both of whom I had the sense and fortune to procure for the Ministry of Works. I said last year that I wished my elevation to the left of the Chair had been due to achievement in the profession and not to the incidental occupancy of ministerial office: and here I am on the right of the Chair. If it is the first time that one of your Members has been so placed, I am the more honoured. I am wondering where this oscillation will lead. It may be, with all due respect to Professor Inglis, to the Chair itself.

Having spoken last year, and not being set on talking at the best of times, although gratified by your invitation, I should have asked the President to excuse me had it not been for the creation of a Ministry of Works, of such interest and concern to you (and to the sister profession of Architecture), and that one of your Members was the first Minister. He ought to have something to say to his own Institution about what has been done, about what is being done, about what is planned, and which, God and other Departments being willing, will or may be done. I was informed yesterday that the Ministry of Works did not advertise itself enough; that few people knew how much had been done and planned in the six months since its creation, and, for the matter of that, by the Office of Works since war started. Even in time of war, even in these times of test and trial, it seems that acts do not always speak for themselves, and that Ministers—Mr. George Hicks and I—ought to have been talking more than we have done, either about these acts or in place of them. Sermons in stones or books in structural steel are not enough.

A new Ministry is not always popular, especially when its creation involves, or should involve, the transfer of authorities and responsibilities from elsewhere; but at least it can be used as a scapegoat of convenient and astonishing capacity.

I will tell you a thing or two it does. In addition to looking after Duck island, in St. James's Park, it is responsible for the provision, maintenance, and repair of 14,000 Government buildings throughout the country. It is itself carrying out an immense building programme, factories of all sorts, storage, landing grounds, hostels, training establishments, camps, depots—more than one million pounds of work a week. It has an office staff of 9,000, half of them technical, and a field force of 12,000. But do not imagine that this is some gargantuan Department about to seize the work of individual engineers and architects. I believe in individuals, and we intend to make full use of them.

One of its achievements has been the substitution for the old priority system, with all its inconsistencies, of a system of allocation in terms of labour to Departments. The amount of work permitted is limited to the capacity of the building industry. It sounds simple; but it has taken months to get it through. Whereas building proposals had reached a peak far in excess of what the industry could meet, we have secured, after vast discussion and negotiation, a reduction to the real capacity of the country, namely, about £350 millions a year. The allocation system will be in operation to-morrow; and all hopefully expect that once it is running there will be a far more efficient building effort. The more urgent construction works will be so manned as to ensure their speedy completion. This has involved much investigation—the more difficult because of the absence of statistics—but when Departments produced their complete programmes some of us were, I think, surprised to find what they had in hand. Returns have been obtained from builders, contractors, and local

authorities, showing the number of employees and the categories of employment and work. But these figures will be seriously incomplete until many local authorities take a more responsible view than they do to-day of their position as large employers of building labour. After three months, less than half have provided the information required: but we shall not stop until we have full and regular statistical control; we are engineers and builders, preferring to move in this field by sight and not by faith.

The Ministry has also established effective control over many building materials, in particular cement and bricks, a cause of much tribulation in the past. In its charter it was invited to institute research into such questions as the adoption of substitutes for building materials, modifications of design and specifications, standardization of design and of all materials for war economy, and to ensure that the results of past and future research are used. All this is done in close collaboration with the Building Research Station; but a Ministry of Works covers a still wider field and there is a great deal of scientific engineering research, in large-scale field experiments and the collation of information from all over the world, which it is now setting out to deal with. In standardization, the policy is to eliminate everything but the minimum necessary for war effort: that may affront the feelings of many engineers and architects; but this is no time to play for safety, nor even to study susceptibilities.

Unnecessary building, unnecessary demolition and clearance, and extravagant reconstruction are being controlled; and licences for building by private interests are now compulsory (and not easily obtainable) for £100 to be spent on any building within twelve months; it may be still further reduced, as nothing that is not contributory to the war should be permitted.

A new Department was established in the Ministry two months ago to help in the rapid repair of damaged houses, services, and factories. It is organized on the basis of Emergency Works Officers at all the important target towns, supervised by Assistant Directors in charge of large grouped areas. There are engineers and architects and contractors, all working together, and from accounts they seem to be carrying out their duties with remarkable, though unadvertised, success.

I will not prolong the tale. But there are two other matters I want to mention: the first because we are aiming at such a combination of all sections and interests of the civil engineering and building industries as will not only notably increase the war effort but also make a radical and permanent improvement in their structure and operation. Discussions were initiated by my Ministry, with the co-operation of the Ministry of Labour, with representatives of the civil engineering and building industries many weeks ago with the view of securing, by their better direction, a more satisfactory building output and a more efficient machine, more in keeping with the serious and urgent needs of the day and better fitted to meet the

post-war problems. This is not a matter that can be arranged by the Ministries of Labour and Works and representatives of the industries alone. Many other Government Departments have to be consulted, and have to be satisfied that their interests will not be prejudiced. We know they will, in fact, be far better served, and for this reason the proposals were made. We are on the threshold of great evolutionary changes. I am much indebted to engineers and architects and to the industries for their co-operation in these explorations and essays: but it is largely my confidence in the practical experience and knowledge of my colleague, Mr. George Hicks, that has encouraged me to go forward with schemes which seem to some to be too venturesome.

On the other matter: a good deal has already been said in public about the planning and reconstruction responsibilities vested in me, to advise on the machinery, constitutional and administrative, necessary for the planning and reconstruction of town and country after the war. This subject also is controversial. Do not let anyone think that what I or anybody else may be doing about the machinery for planning detracts from the war effort. I said in another place that the idea of a planned and ordered reconstruction is surely an incentive to and an encouragement of war effort: and surely engineers, of all people, so careful in planning their own works, should welcome planning in this larger sphere. They should, in fact, be among those who, insisting on a proper design of whatever they are about to build, must welcome a design for living not only in planned and ordered communities of concrete and bricks and timber and stone and steel, but also of highways and byways; of farms where farms should be, and flowers and grass and trees where they should be; and of industrial communities where they should be (and definitely not where they should not be). There must be co-ordination between living and working and moving and playing, with amenities, natural and otherwise, of civilized life, instead of the haphazard, confused disorder and inconvenience of our lives, or the monstrous and obscene mutilations of the countryside.

In this connexion I welcome contacts with the President and Sir Clement Hindley, and I congratulate The Institution on the steps taken to investigate many of the post-war problems of concern to engineers, whilst we all welcome the establishment of the new committee announced to-day.

One word to engineers from my own experience in the profession. The Institution has done well to initiate and finance a course of instruction at Cambridge University under the distinguished Professor of Engineering there, in order that the engineer may be something more than an engineer —which, in fact, he too rarely is—and that he may have some idea of the general problems of management and of the broader issues involved in engineering works and in business generally.

On the moral issues of war and peace we in this country are on un-

assailable ground, and we know that there is no compromise possible. We know, too, that we have opportunities of immense service to mankind and to the world. We may be fighting for self-preservation with no bridge of escape, and desiring none, but beyond self-survival there is this opportunity of something far bigger than ourselves, in the conquest of evil and in the establishment of a better order here and throughout the world, for a better order here will depend on a better order everywhere. Put quite simply, I conceive that we are engaged in a struggle for the standard of living, economic and cultural, throughout the world. This is not a purely materialistic end; it includes and transcends the materialistic. A rise in the standard of living, if economic alone, would at best be static, and what was gained would soon be lost. Of at least equal importance is a rise in, and the permanent establishment of, the moral and spiritual standards of living, all now in the balance; and this for all peoples in all countries. And it will be for us to see that, when peace falls like a benediction on the world, it bestows for all time and for all people security and happiness, and freedom from fear and want.

Your Chairman has made an announcement about Greece. In what I have said about the establishment of moral and spiritual values I see —and I think we all see—in Greece the faith for which the Empire fights, and for which we look beyond war, as an engineer looks beyond this day to the completion of the job, to the passage of traffic over his mighty bridge, to the first flow of water over the spillway of his dam. Life and war are all engineering achievements after a kind. If there were more of the engineer's outlook, his factor of safety; his factor of efficiency, his planned organization, there would be fewer disasters and difficulties in both life and war. But difficulties and disasters there must be, and bridges have been known to collapse more than once; you all know one bridge that I am talking about. Do let us see these Mediterranean happenings, particularly in Greece, in their right perspective, as an engineer surely would. There are setbacks to be encountered on every job, however carefully planned; but it is an essential of engineering, having counted the cost, to take the risk. Here in Greece was an obvious and admitted risk, but every dictate of honour and moral obligation proclaimed that the risk must be taken; and it was taken, nobly and bravely. If to us, gentlemen, as engineers, this is a week when the progress report is not quite so satisfactory as it might be, we are used to that, and we look to the end of the job.

I give you the good health of your Institution. May it flourish and continue to command the respect of the community, as in times past.

Sir Leopold Savile, K.C.B., President of The Institution, who responded, said: I have first of all to thank Lord Reith, the first civil engineer to occupy the post of a Minister of the Crown, for coming here and speaking as he has done. He has done us great honour, and what he has said must be an inspiration to the profession as a whole.

I do not intend to attempt to make a speech: I am here only as a sandwich between one speaker and those who are to follow. But there are a few matters which I feel it my duty to refer to, in the nature of a résumé of the work of The Institution during the past session. We have been able to hold meetings monthly instead of fortnightly, and we have had many interesting Papers, for one of which the Council have awarded the Baker Gold Medal to Dr. Oscar Faber, whilst in spite of the war the attendances have been good. The Institution Journal has been published nonthly through the session, and I think that all members will agree that the change from the old "Minutes of Proceedings" to the "Journal" nas been a great success and of considerable value. It must be useful to nembers, especially at the present time, to have an up-to-date record of our activities instead of the old "Minutes," which were published quite a year after the events which they recorded.

The Council have given consideration, as Lord Reith has mentioned, o post-war development, and several committees have been working on in investigation of the different aspects of this question and have been keeping in touch with the Ministry of Works and Buildings in connexion herewith. I think we all realize the importance of having schemes prepared in advance so that they can be implemented when the proper ime comes, and we feel that this is one of the lines upon which The Institution may be of assistance to the Ministry.

With regard to the technical education of engineers, there are two teps in which the Council have been able to co-operate with Government Departments. The first is that a Student liable for service on reaching he age of 18 may have his calling-up deferred on production of a certificate rom the Secretary of The Institution to the effect that he is satisfied that he student is working in the capacity of an engineering learner and has nade arrangements to study for the Associate Membership examination. The second is that those already serving with the colours can take advantage of vocational correspondence courses on the subjects of the Associate Membership examination, arranged for by the War Office Executive and the Education authorities, and it is hoped that by means f these two schemes men may be able to pass the examination before or uring their war service, and so avoid finding themselves at the end of the var without any technical qualifications, as was the case after the last war.

Lord Reith has mentioned that during the past session the Council pproached the Vice-Chancellor of Cambridge University with an offer o provide the necessary finance for the establishment of a lectureship on he economics of engineering design and construction and also management, heir desire being to assist in the aim of eventually including these subjects n the curriculum of the Mechanical Science Tripos. This offer has been ery favourably received by the University authorities, and there is every rospect of making a start by forming a panel of eminent engineers who vill give lectures during next winter on these subjects.

Lord Reith has also mentioned the Advisory Committee. You will have no doubt seen it announced in *The Times* this morning that the Lord President of the Council, with the approval of the Prime Minister, has arranged an Engineering Advisory Committee, under the Chairmanship of Lord Hankey, to advise the Government upon engineering questions connected with the war effort. In this connexion, you may be interested to know that this was first proposed by my friend, Mr. J. R. Beard, President of the Institution of Electrical Engineers, at a meeting of the Engineering Joint Council last December, with the result that, with the approval of the Councils of the eight constituent Institutions, a letter signed by their Presidents was written to the Prime Minister, suggesting the formation of such a Committee. The proposal was received favourably, and interviews were held with the view of defining the details and terms of reference and obtaining suggestions in regard to the construction of the proposed Committee. The Government were anxious that this Committee should not necessarily represent any particular Institution, but rather the various branches of the field of engineering covered. The original idea was that the Committee should act in parallel with the Scientific Advisory Committee—also under the Chairmanship of Lord Hankey—and in some sense form a liaison between that Committee and the producers. But you will see, if you read the terms of reference, that this has been expanded considerably. I think that this is the first time that a body of engineers has been formed at the request of the Government to advise them on engineering matters, and it is hoped and believed that such an Advisory Committee will be of value not only during the war but also in connexion with post-war development. Mr. Beard is to be congratulated on having started an idea which has reached fruition in the way that this has done.

Finally, I should like to express my own and the Council's appreciation of the work of our Secretary, Mr. Graham Clark, and his staff during the very difficult period. In spite of the fact that the staff has been considerably reduced, the essential part of the work of The Institution has been carried on, I think you will agree, in a most satisfactory manner, and I consider that the gratitude of all members is due to the staff for what has been done during the past session. I will only add my thanks to Lord Reith for having come here to-day, and for the speech with which he has favoured us.

Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S., Vice-President of The Institution, said : It is with great pleasure that, with a brevity which must not be mistaken for lack of subject matter, I rise to propose the health of "The Guests." To one and all we wish to extend a most cordial welcome, and we regard it as a very great compliment to our Institution that, notwithstanding their multifarious preoccupations, so many distinguished individuals have found time to participate in our conviviality; albeit I was going to say that that hospitality has had to be pitched in a somewhat

minor key, I think, in view of the ample fare that has been provided by Grosvenor House, such an apology is now scarcely necessary. It not infrequently happens that the toast of "The Guests" is introduced by a speech drawn up rather on the lines of an auction sales catalogue, in which the goods exhibited, whether ancient or modern, have their merits extolled with a degree of veracity characterized perhaps more by elasticity than absolute rigidity. If for no better reason, time debars me from attempting that form of treatment, and in my survey of our exalted guests I will ask you to view the landscape rather in terms of mountain ranges than individual peaks. But inasmuch as he comes into a classification by himself, I should like, on your behalf, to give a special greeting to the Dean of Westminster, who by his presence here imparts a dignity and, perhaps I might almost say, a respectability to our proceedings which we value very highly. In these days he must have many anxious moments, and on that account our good wishes go out to him in most abundant measure, not merely for his own personal welfare, but also for the welfare of that great, historic, and priceless edifice which is entrusted to his charge.

In the application to engineering of science in its most recondite forms we have very worthy representatives at this gathering in Sir Edward Appleton, Sir Frank Smith, and Sir William Bragg. Amongst his numerous researches, Sir William has certainly made one discovery which is a source of envy to many of us seniors—he most obviously has discovered the secret of perpetual youth both in its intellectual and in its physical aspects. Architecture and engineering are kindred subjects so closely interlocked that we are very glad indeed to have among us Sir James West, the chief architect, and Mr. George Hicks, the Parliamentary Secretary to the Ministry of Works and Buildings. We are also exceedingly pleased to have with us that most eminent and world-renowned architect, Sir Edwin Lutyens. Even out of war good may emerge, and in the days to come a soul-stirring vista of reconstruction is opening out in which architects and engineers will have a unique opportunity of co-operating and of developing that *amicabilis concordia* which is so eminently desirable and which has been referred to so eloquently by Lord Reith.

I think that at this stage I ought to refer to our good friend Mr. Hugh Beaver. Mr. Beaver in his activities is so intimately associated with our Institution that it is difficult to regard him as a guest, but I force myself on this occasion to do so in order that it may give us an opportunity of congratulating him upon his recent important Government appointment as Director-General of Works and Buildings.

As the senior of the engineering Institutions we are, of course, always very glad to see our children gathered round our fireside, and at this family party we are particularly pleased to welcome Mr. Stanier, President of The Institution of Mechanical Engineers, Mr. Beard, President of The Institution of Electrical Engineers, and Lord Stonehaven, President of The Institution of Naval Architects.

The Civil Service has presented us with two very distinguished guests in the persons of Sir John Maude, Permanent Secretary to the Ministry of Health, and Sir Leonard Browett, Permanent Secretary to the Ministry of Transport. Owing to enemy action these Ministries have had thrust upon them many new activities and responsibilities, which they have faced up to with great determination and which bring them into the closest possible contact with engineers of every type and denomination. We are also very pleased to welcome the London Regional Commissioner, Sir Ernest Gowers. I do not know exactly under what category I ought to bring him, but we all know that when necessity arises a Regional Commissioner is invested with almost dictatorial powers. It is a position of very great responsibility, in which high administrative ability has to be combined with a proper appreciation of the military situation. That brings me naturally to the last mountain range which I want to indicate to you in my survey—the fighting forces. We are very gratified and honoured by the presence of Air-Marshal Sir Christopher Courtney. He is the member of the Air Council who is particularly responsible for supply, and one of the commodities with which he is concerned is aerodromes. The vigour which he has shown in that connexion must be obvious even to the most shortsighted observer, because it would be indeed difficult to find any residential district in this country which could not be described as a " near miss " in relation to an aerodrome. A close and ever closer co-operation between military engineers and civil engineers has been the aim and ambition of our Institution for many years. Consequently we extend a particular welcome to two very distinguished military engineers, Major-General King, Chief Engineer of the Home Forces, and Major-General Cave-Browne, Director of Fortifications and Works. The Navy is most worthily represented by Admiral Sir Edward Evans. In his early days Sir Edward was an intrepid Antarctic explorer, and he has many other gallant actions to his credit. Perhaps one of the most gallant is that on two occasions he was Lord Rector of Aberdeen University! On those occasions the students are apt to be rather more enthusiastic than courteous, but at any rate, as might be expected, from the lady students he obtained a very welcome reception. He tells me that that welcome was put into rhyme by the chief lady student, to this effect (she was speaking of the Navy):

" Their arms are our defence,
Our arms their recompense.
Ladies, fall in."

Now it only remains for me to couple this toast with the name of Captain Hudson, who has kindly consented to reply. Captain Hudson, after holding many important Parliamentary posts, is now the Civil Lord of the Admiralty. To a mere landlubber such as myself the composition of the Board of Admiralty is an unfathomable and awe-inspiring mystery.

There are Lords of the first order of magnitude and of the second order' there are Sea Lords and, in contradistinction I suppose, our distinguished guest might be regarded as a land-lord. But I prefer to regard him as the power on high who directs the civil activities of the Navy, and in that capacity in particular his presence is most welcome and appropriate because, as you all know, our President, Sir Leopold Savile, won his spurs on the field of civil engineering related to the Navy.

With that brief survey, I ask you to rise and drink to the health of the guests, whether they come as guests of The Institution as a whole or as guests of private members. We extend to them all a most cordial greeting.

Captain Austin Hudson, M.P., Civil Lord of the Admiralty, said: It is my very pleasant duty to return thanks on behalf of the guests, and I think that on this occasion we guests have to thank you doubly; first of all for the most excellent lunch which we have all enjoyed—and in these days it is quite the exception to be asked out to lunch—and secondly, for giving us the opportunity of listening to the very excellent speeches of Lord Reith and others. I confess that, as a member of the Board of Admiralty who is responsible for works and buildings, I was somewhat apprehensive when I heard of the formation, or proposed formation, of the new Ministry. I, like other members of the Services present, and of other Government Departments, have enough trouble as it is in persuading the Treasury that any works which I put forward are both economical and sound, as, of course, they are, and I did not relish yet another fence to surmount. Furthermore, the Ministry of Works and Buildings was formed from the old Office of Works. The Office of Works, to my mind, was connected with ancient monuments, so that I was very much afraid that the new Minister of Works and Buildings would be a sort of glorified ancient monument himself. You will agree, after what we have heard from Lord Reith, that he gets away from that criticism at any rate. I am glad also to say that my fears proved to be ill-founded and that our relations with the new Ministry continue to be friendly. In fact, I have found them—and I am certain I can speak for other Departments—both helpful and sympathetic in carrying out a very difficult task. Lord Reith has mentioned that he has taken on what I consider a very thankless duty, that of trying to adjust the works programmes of the various Departments to meet the existing labour forces. I knew that this was coming about, and I was very suspicious as to what the result would be. When, however, I found that virtue was rewarded and that the Admiralty had been cut less than some other Departments I realized that Lord Reith was a man of great sagacity and common sense.

In one respect I pity him and his new Ministry. I think many of us hope that when the war is over we shall be able to sit back a little and take things rather more easily. But when that moment arrives Lord Reith will have to turn to his Department and say, " Now, boys, this is when

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we really begin." In that field of reconstruction he has my sincere sympathy.

I cannot conclude my brief remarks without saying how much we, at the Admiralty, appreciate the help given and offered by the members of your Institution. It is given in a number of ways, and it is invaluable. We are very grateful indeed for it. On behalf of the guests I thank you for your hospitality to-day and for providing a very delightful interlude in a hard day's work.

The proceedings then terminated.